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Geosynthetic Design and Construction Guidelines

Participant Notebook

National Highway Institute

NOTICE

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16. ABSTRACT

This manual is an updated version of the Geotextile Design & Construction Guidelines, used for the FHWA training course Geosynthetic Engineering Workshop. The update was performed to reflect current practices and codes for geotextile design, and has been expanded to address geogrid and geomembrane materials. The manual was prepared to enable the Highway Engineer to correctly identify and evaluate potential applications of geosynthetics as an alternative to other construction methods and as a means to solve construction problems. With the aid of this text, the Highway Engineer should be able to properly design, select, test, specify, and construct with geotextiles, geocomposite drains, geogrids and related materials in drainage, sediment control, erosion control, roadway, and embankment on soft soils applications. Steepened slope and retaining wall applications also are addressed, but designers are referred to the FHWA Demonstration Project No. 82 references on mechanically stabilized earth structures for details on design. Application of geomembranes and other barrier materials to highway works are summarized within. This manual is directed toward geotechnical, hydraulic, roadway, bridge and structures, and route layout highway engineers.

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SI CONVERSION FACTORS				
APPROXIMATE CONVERSIONS FROM SI UNITS				
Symbol	**When You Know**	**Multiply By**	**To Find**	**Symbol**
		LENGTH		
mm	millimeters	0.039	inches	in
m	meters	3.28	feet	ft
m	meters	1.09	yards	yd
km	kilometers	0.621	miles	mi
		AREA		
mm^2	square millimeters	0.0016	square inches	in^2
m^2	square meters	10.764	square feet	ft^2
m^2	square meters	1.195	square yards	yd^2
ha	hectares	2.47	acres	ac
km^2	square kilometers	0.386	square miles	mi^2
		VOLUME		
ml	millimeters	0.034	fluid ounces	fl oz
l	liters	0.264	gallons	gal
m^3	cubic meters	35.71	cubic feet	ft^3
m^3	cubic meters	1.307	cubic yards	yd^3
		MASS		
g	grams	0.035	ounces	oz
kg	kilograms	2.202	pounds	lb
		TEMPERATURE		
°C	Celsius	1.8 C + 32	Fahrenheit	°F
		WEIGHT DENSITY		
kN/m^3	kilonewton / cubic meter	6.36	poundforce / cubic foot	pcf
		FORCE and PRESSURE or STRESS		
N	newtons	0.225	poundforce	lbf
kN	kilonewtons	225	poundforce	lbf
kPa	kilopascals	0.145	poundforce / square inch	psi
kPa	kilopascals	20.9	poundforce / square foot	psf

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APRIL 1998

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Preface

The 1998 update to the Geosynthetic Design & Construction Guidelines manual was initiated to incorporate the following recent publications:

- *the 1997 revised AASHTO Standard Specifications for Geotextiles — M 288;*
- *the 1997 interims to the AASHTO Standard Specifications for Highway Bridges, Sixteenth Edition, 1996;*
- *Mechanically Stabilized Earth Walls and Reinforced Soil Slopes Design and Construction Guidelines, FHWA Demonstration Project 82, August 1997; and*
- *Corrosion / Degradation of Soil Reinforcements for Mechanically Stabilized Earth Walls and Reinforced Soil Slopes, FHWA Demonstration Project 82, August 1997.*

The 1995 Geosynthetic Design & Construction Guidelines manual has evolved from the following FHWA manuals:

- *Geotextile Design & Construction Guidelines - Participant Notebook by Barry R. Christopher and Robert D. Holtz; STS Consultants, Northbook, Illinois, and GeoServices, Inc., Boca Raton, Florida; October 1988 and selectively updated to April 1992.*
- *Geotextile Engineering Manual by Barry R. Christopher and Robert D. Holtz; STS Consultants, Northbrook, Illinois; March, 1985; 917 p.*
- *Use of Engineering Fabrics in Transportation Type Related Applications by T. Allan Haliburton, J.D. Lawmaster, and Verne C. McGuffey; 1981.*

This 1995 Geosynthetic Design & Construction Guidelines manual was also derived from the following FHWA reports:

- *Guidelines for Design, Specification, and Contracting of Geosynthetic Mechanically Stabilized Earth Slopes on Firm Foundations; by Ryan R. Berg; Ryan R. Berg & Associates, St. Paul, Minnesota; January, 1993; 88p.*
- *Reinforced Soil Structures - Volume I, Design and Construction Guidelines, and Volume II Summary of Research and Systems Information; by B.R. Christopher, S.A. Gill, J.P. Giroud, J.K. Mitchell, F. Schlosser, and J. Dunnicliff; STS Consultants, Northbrook, Illinois, November 1990.*

Special Acknowledgement

Jerry A. DiMaggio, P.E. is the FHWA Technical Consultant for this work, and served in the same capacity for most of the above referenced publications. Mr. DiMaggio's guidance and input to this and the previous works was invaluable.

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1.0 INTRODUCTION, IDENTIFICATION, AND EVALUATION

1.1 BACKGROUND

This manual was prepared to assist design engineers, specification writers, estimators, construction inspectors, and maintenance personnel with the design, selection, and installation of geosynthetics. In addition to providing a general overview of these materials and their applications, step-by-step procedures are given for the cost-effective use of geosynthetics in drainage and erosion control systems, roadways, and reinforcement, and in containment applications. Although the title refers to the general term *geosynthetic*, specific applications address the appropriate use of subfamilies of geotextiles, geogrids, geocomposites, and geomembranes.

The basis for much of this manual is the FHWA *Geotextile Engineering Manual* (Christopher and Holtz, 1985). Other sources of information include the books by Koerner (1994), John (1987), and Veldhuijzen van Zanten (1986). If you are not already familiar with geosynthetics, you are encouraged to read Richardson and Koerner (1990) and Ingold and Miller (1988), especially if you are attempting to use geosynthetics for the first time. A listing of other geosynthetics literature can be found in Cazzuffi and Anzani (1992) and Holtz and Paulson (1988), both of which are reproduced in Appendix A. Comprehensive geosynthetic bibliographies have recently been prepared by Giroud (1993, 1994). If you are unfamiliar with geosynthetics terminology, see ASTM (1997) D 4439 Standard Terminology for Geosynthetics. Basic terms are defined in Appendix B. The authors assume that you are already familiar with the engineering basics of geotechnical, highway, hydraulic, retaining wall, and pavement design. Common notation and symbols are used throughout this manual, and a list is provided in Appendix C for easy reference. These notations and symbols are generally consistent with the International Geosynthetic Society's (IGS) Recommended Mathematical and Graphical Symbols (1993).

Sample specifications included in this manual were developed in several cases by Task Force 25 Subcommittee of the American Association of State Highway and Transportation Officials (AASHTO, 1990), the Association General Contractors (AGC), and the American Road and Transportation Builders Associations (ARTBA) Joint Committee, along with representatives from the geosynthetic industry. Important input has also been obtained from the AASHTO-AGC-ARTBA Task Force 27 Subcommittee (1990). Specifications from the FHWA Guidelines for Design, Specification, and Contracting of Geosynthetic Mechanically Stabilized Earth Slopes on Firm Foundations (Berg, 1993) are also used with this manual. Finally, sample specifications were obtained from some state Departments of Transportation. **These specifications are meant to serve only as guidelines and should be modified as required by engineering judgment and experience, based upon project specific design and performance criteria.**

Chapter 1 introduces you to the functions and applications of geosynthetics, to the identification of the materials, and to the methods used to evaluate their properties. The remaining nine chapters give specific details about important application categories of geosynthetics, such as drainage and roadways. Each chapter provides a systematic approach to applying geosynthetics so that successful cost-effective designs and installations can be achieved.

1.2 DESIGN APPROACH

We recommend the following approach to designing with geosynthetics:

1. Define the purpose and establish the scope of the project.
2. Investigate and establish the geotechnical conditions at the site (geology, subsurface exploration, laboratory and field testing, etc.).
3. Establish application criticality, severity, and performance criteria. Determine external factors that may influence the geosynthetic's performance.
4. Formulate trial designs and compare several alternatives.
5. Establish the models to be analyzed, determine the parameters, and carry out the analysis.
6. Compare results and select the most appropriate design; consider alternatives versus cost, construction feasibility, etc. Modify the design if necessary.
7. Prepare detailed plans and specifications including: a) specific property requirements for the geosynthetic; and b) detailed installation procedures.
8. Hold preconstruction meeting with contractor and inspectors.
9. Approve geosynthetic on the basis of specimens' laboratory test results and/or manufacturer's certification.
10. Monitor construction.
11. Inspect after events (*e.g.*, 100 year rainfall) that may tax structure performance.

By following this systematic approach to designing with geosynthetics, cost-effective designs can be achieved, along with improved performance, increased service life, and reduced maintenance costs. Good communication and interaction between all concerned parties is imperative throughout the design and selection process.

1.3 DEFINITIONS, MANUFACTURING PROCESSES, AND IDENTIFICATION

ASTM (1997) has defined a *geosynthetic* as a planar product manufactured from a polymeric material used with soil, rock, earth, or other geotechnical-related material as an integral part of a civil engineering project, structure, or system. A *geotextile* is a permeable geosynthetic made of textile materials. There are a number of other materials available today that technically are not textiles -- including webs, grids, nets, meshes, and composites -- that are used in combination with or in place of geotextiles. These are sometimes referred to as geotextile-related materials. Geotextiles and related materials all fall under the principal category of geosynthetics. *Geogrids*, geosynthetics primarily used for reinforcement, are formed by a regular network of tensile elements with apertures of sufficient size to interlock with surrounding fill material. *Geomembranes* are low-permeability geosynthetics used as fluid barriers. Geotextiles and related products, such as nets and grids, can be combined with geomembranes and other synthetics to complement the best attributes of each material. These products are called *geocomposites*, and they may be geotextile-geonets, geotextile-geogrids, geotextile-geomembranes, geomembrane-geonets, geotextile-polymeric cores, and even three-dimensional polymeric cell structures. There is almost no limit to the combinations of geocomposites.

A convenient classification scheme for geosynthetics is provided in Figure 1-1. For details on the composition, materials, and manufacturing processes, see Koerner (1994), Ingold and Miller (1988), Veldhuijzen van Zanten (1986), Christopher and Holtz (1985), Giroud and Carroll (1983), Rankilor (1981), and Koerner and Welsh (1980). Most geosynthetics are made from synthetic polymers of polypropylene, polyester, or polyethylene. These polymer materials are highly resistant to biological and chemical degradation. Less-frequently-used polymers include polyamides (*nylon*) and glass fibers. Natural fibers, such as cotton, jute, etc., could also be used as geotextiles, especially for temporary applications, but they have not been researched or utilized in the U.S. as widely as polymeric geotextiles.

In manufacturing geotextiles, elements such as fibers or yarns are combined into planar textile structures. The fibers can be continuous *filaments*, which are very long thin strands of a polymer, or *staple fibers,* which are short filaments, typically 20 to 150 mm long. The fibers may also be produced by slitting an extruded plastic sheet or film to form thin flat tapes. In both filaments and slit films, the extrusion or drawing process elongates the polymers in the direction of the draw and increases in the filament strength.

Geotextile type is determined by the method used to combine the filaments or tapes into the planar structure. The vast majority of geotextiles are either *woven* or *nonwoven.* Woven geotextiles are made of *monofilament*, *multifilament*, or *fibrillated* yarns, or of slit films and tapes. The weaving process is as old as Homo sapiens' textile cloth-making. Nonwoven textile manufacture is a modern development, a *high-tech* process industry, in which synthetic polymer fibers or filaments are continuously extruded and spun, blown or otherwise laid onto a moving belt. Then the mass of filaments or fibers are either *needlepunched*, in which the filaments are mechanically entangled by a series of small needles, or *heat bonded,* in which the fibers are *welded* together by heat and/or pressure at their points of contact in the nonwoven mass.

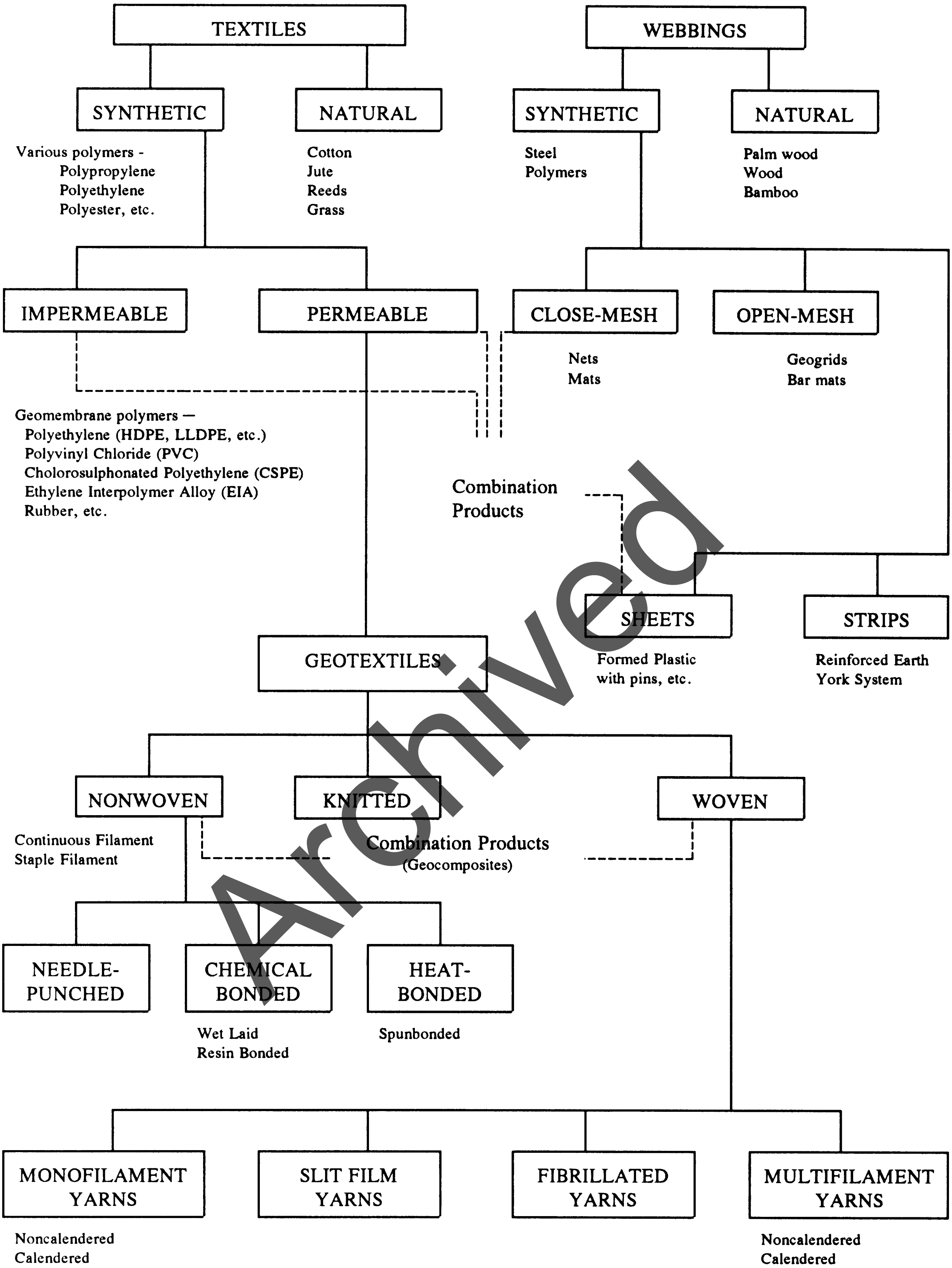


Figure 1-1 Classification of geosynthetics and other soil inclusions (after Rankilor, 1981).

The manufacture of geotextile-related products is as varied as the products themselves. Geonets, geosynthetic erosion mats, geogrids, etc., can be made from large and often rather stiff filaments formed into a mesh with integral junctions or which are welded or glued at the crossover points. Geogrids with integral junctions are manufactured by extruding and orienting sheets of polyolefins. These types of geogrids are usually called *stiff* geogrids. Geogrids are also manufactured of polyester yarns, joined at the crossover points by a knitting or weaving process, and encased with a polymer-based, plasticized coating. These types of geogrids are generally called *flexible* geogrids. Manufacture of geomembranes and other geosynthetic barriers is discussed in Chapter 10.

Geocomposites result when two or more materials are combined in the geosynthetic manufacturing process. Most are used in highway drainage applications and waste containment. A common example of a geocomposite is a prefabricated drain formed by wrapping a fluted or dimpled polymeric sheet, which acts as a conduit for water, with a geotextile which acts as a filter.

Geosynthetics are generically identified by:

1. polymer (descriptive terms, *e.g.*, high density, low density, etc. should be included);
2. type of element (*e.g.*, filament, yarn, strand, rib, coated rib), if appropriate;
3. distinctive manufacturing process (*e.g.*, woven, needlepunched nonwoven, heatbonded nonwoven, stitchbonded, extruded, knitted, roughened sheet, smooth sheet), if appropriate;
4. primary type of geosynthetic (*e.g.*, geotextile, geogrid, geomembrane, etc.);
5. mass per unit area, if appropriate (*e.g.*, for geotextiles, geogrids, GCLs, erosion control blankets,) and/or thickness, if appropriate (*e.g.*, for geomembranes); and
6. any additional information or physical properties necessary to describe the material in relation to specific applications.

Four examples are:

- polypropylene staple filament needlepunched nonwoven geotextile, 350 g/m^2;
- polyethylene geonet, 440 g/m^2 with 8 mm openings;
- polypropylene extruded biaxial geogrid, with 25 mm x 25 mm openings; and
- high-density polyethylene roughened sheet geomembrane, 1.5 mm thick.

1.4 FUNCTIONS AND APPLICATIONS

Geosynthetics have six primary functions:

1. filtration
2. drainage

3. separation
4. reinforcement
5. fluid barrier, and
6. protection

Geosynthetic applications are usually defined by their primary, or principal, function. For example, geotextiles are used as *filters* to prevent soils from migrating into drainage aggregate or pipes, while maintaining water flow through the system. They are similarly used below riprap and other armor materials in coastal and stream bank protection systems to prevent soil erosion.

Geotextiles and geocomposites can also be used as *drainage*, or lateral transmission media, by allowing water to drain from or through soils of lower permeability. Geotextile applications include dissipation of pore water pressures at the base of roadway embankments. For situations with higher flow requirements, geocomposite drains have been developed. These materials are used as pavement edge drains, slope interceptor drains, and abutments and retaining wall drains. Filtration and drainage are addressed in Chapter 2 - Geosynthetics in Subsurface Drainage Systems.

Geotextiles are often used as *separators* to prevent road base materials from penetrating into the underlying soft subgrade, thus maintaining the design thickness and roadway integrity. Separators also prevent fine-grained subgrade soils from being pumped into permeable, granular road bases. Separators are discussed in Chapter 5 - Geosynthetics in Roadways and Pavements.

Geogrids and geotextiles can also be used as *reinforcement* to add tensile strength to a soil matrix, thereby providing a more competent structural material. Reinforcement enables embankments to be constructed over very soft foundations and permits the construction of steep slopes and retaining walls. Reinforcement applications are presented in Chapter 7 - Reinforced Embankments on Soft Foundations; Chapter 8 - Reinforced Slopes; and Chapter 9 - Mechanically Stabilized Earth Retaining Walls and Abutments.

Geomembranes, thin-film geotextile composites, geosynthetic clay liners, and field-coated geotextiles are used as *fluid barriers* to impede the flow of a liquid or gas from one location to another. This geosynthetic function has wide application in asphalt pavement overlays, encapsulation of swelling soils, and waste containment. Barrier applications are summarized in Chapters 6 - Pavement Overlays, and 10 - Geomembranes and Other Geosynthetic Barriers.

In the sixth function, *protection*, the geosynthetic acts as a stress relief layer. Temporary geosynthetic blankets and permanent geosynthetic mats are placed over the soil to reduce erosion caused by rainfall impact and water flow shear stress. A protective cushion of nonwoven geotextiles is often used to prevent puncture of geomembranes (by reducing point stresses) from stones in the adjacent soil or drainage aggregate during installation and while in service. Erosion control is presented in Chapter 3 - Geotextiles in Riprap Revetments and Other Permanent Erosion Control Systems; and Chapter 4 - Temporary Runoff and Sediment Control.

In addition to the primary function, geosynthetics usually perform one or more *secondary* functions. The primary and secondary functions make up the total contribution of the geosynthetic to a particular application. A listing of common applications according to primary and secondary functions is presented in Table 1-1. It is important to consider both the primary and secondary functions in the design computations and specifications.

1.5 EVALUATION OF PROPERTIES

Today, there are more than 600 different geosynthetic products available in North America. Because of the wide variety of products available, with their different polymers, filaments, weaving (or nonwoven) patterns, bonding mechanisms, thicknesses, masses, etc., they have a considerable range of physical and mechanical properties. Thus, the process of comparison and selection of geosynthetics is not easy. Geosynthetic testing has progressed significantly since the first Geotextile Engineering Manual (Christopher and Holtz, 1985) was published. Specific test procedures are given in AASHTO (1997), ASTM (1997), and GRI (1997). The AASHTO Standard Specification for Geotextiles, designated M 288, is specifically for highway applications and addresses subsurface drainage, sediment control, erosion control, separation, and pavement overlay applications. The AASHTO M 288 specification can be found in Appendix D. Testing procedures developed by the Geosynthetics Research Institute of Drexel University are considered only interim standards until an equivalent ASTM standard is adopted. ASTM and GRI standards are listed in Appendix E.

The particular, required design properties of the geosynthetic will depend on the specific application and the associated function(s) the geosynthetic is to provide. The properties listed in Table 1-2 cover the range of important criteria and properties required to evaluate geosynthetic suitability for most applications. It should be noted that not all of the listed requirements will be necessary for all applications. For a specific application requirements, refer to the subsequent chapter covering that application.

All geosynthetic properties and parameters to be considered for specific projects are listed in Table 1-3. Again, see AASHTO (1997), ASTM (1997), and GRI (1997) for test procedures for each specific property. Manufacturers can provide information on general properties. The December issue of *Geotechnical Fabrics Report* magazine, published by the Industrial Fabrics Association

TABLE 1-1
REPRESENTATIVE APPLICATIONS AND CONTROLLING FUNCTIONS OF GEOSYNTHETICS

PRIMARY FUNCTION	APPLICATION	SECONDARY FUNCTION(S)
Separation	Unpaved Roads (temporary & permanent)	Filter, drains, reinforcement Filter, drains
	Paved Roads (secondary & primary)	Filter, drains, reinforcement
	Construction Access Roads	Filter, drains, reinforcement
	Working Platforms	Filter, drains, reinforcement
	Railroads (new construction)	Filter, drains, reinforcement
	Railroads (rehabilitation)	Reinforcement, drains, protection
	Landfill Covers	Reinforcement, drains
	Preloading (stabilization)	Filter, drains, reinforcement
	Marine Causeways	Filter, drains, reinforcement
	General Fill Areas	Filter, drains, reinforcement
	Paved & Unpaved Parking Facilities	Filter, drains, reinforcement
	Cattle Corrals	Filter, drains, reinforcement
	Coastal & River Protection	Filter, drains, protection
	Sports Fields	
Filter	Trench Drains	Separation, drains
	Pipe Wrapping	Separation, drains, protection
	Base Course Drains	Separation, drains
	Frost Protection	Separation, drainage, reinforcement
	Structural Drains	Separation, drains
	Toe Drains in Dams	Separation, drains
	High Embankments	Drains
	Filter Below Fabric-Form	Separation, drains
	Silt Fences	Separation, drains
	Silt Screens	Separation
	Culvert Outlets	Separation
	Reverse Filters for Erosion Control:	Separation
	Seeding and Mulching	
	Beneath Gabions	
	Ditch Armoring	
	Embankment Protection, Coastal	
	Embankment Protection, Rivers & Streams	
	Embankment Protection, Lakes	
	Vertical Drains (wicks)	
Drainage-Transmission	Retaining Walls	Separation, filter
	Vertical Drains	Separation, filter
	Horizontal Drains	Reinforcement
	Below Membranes (drainage of gas & water)	Reinforcement, protection
	Earth Dams	Filter
	Below Concrete (decking & slabs)	Protection

TABLE 1-1
REPRESENTATIVE APPLICATIONS AND
CONTROLLING FUNCTIONS OF GEOSYNTHETICS
(continued)

PRIMARY FUNCTION	APPLICATION	SECONDARY FUNCTION(S)
Reinforcement	Pavement Overlays	----------
	Subbase Reinforcement in Roadways & Railways	Filter
	Retaining Structures	Drains
	Membrane Support	Separation, drains, filter, protection
	Embankment Reinforcement	Drains
	Fill Reinforcement	Drains
	Foundation Support	Drains
	Soil Encapsulation	Drains, filter, separation
	Net Against Rockfalls	Drains
	Fabric Retention Systems	Drains
	Sand Bags	----------
	Reinforcement of Membranes	Protection
	Load Redistribution	Separation
	Bridging Nonuniformity Soft Soil Areas	Separation
	Encapsulated Hydraulic Fills	Separation
	Bridge Piles for Fill Placement	----------
Fluid Barrier	Asphalt Pavement Overlays	----------
	Liners for Canals and Reservoirs	----------
	Liners for Landfills and Waste	----------
	Repositories	----------
	Covers for Landfill and Waste	----------
	Repositories	----------
	Cutoff Walls for Seepage Control	----------
	Waterproofing Tunnels	----------
	Facing for Dams	----------
	Membrane Encapsulated Soil Layers	----------
	Expansive Soils	----------
	Flexible Formwork	----------
Protection	Geomembrane cushion	Drains
	Temporary erosion control	Fluid barrier
	Permanent erosion control	Reinforcement, fluid barrier

TABLE 1-2
IMPORTANT CRITERIA AND PRINCIPAL PROPERTIES REQUIRED FOR GEOSYNTHETIC EVALUATION

CRITERIA AND PARAMETER	PROPERTY[1]	FUNCTION					
		Filtration	Drainage	Separation	Reinforcement	Barrier	Protection
Design Requirements:							
Mechanical Strength							
Tensile Strength	Wide Width Strength	–	–	–	✔	✔	–
Tensile Modulus	Wide Width Modulus	–	–	–	✔	✔	–
Seam Strength	Wide Width Strength	–	–	–	✔	✔	–
Tension Creep	Creep Resistance	–	–	–	✔	✔	–
Compression Creep	Creep Resistance	–	✔[2]	–	–	–	–
Soil-Geosynthetic Friction	Shear Strength	–	–	–	✔	✔	✔
Hydraulic							
Flow Capacity	Permeability	✔	✔	✔	✔	✔	–
	Transmissivity	–	✔	–	–	–	✔
Piping Resistance	Apparent Opening Size	✔	–	✔	✔	–	✔
Clogging Resistance	Porimetry	✔	–	–	–	–	✔
	Gradient Ratio or Long-Term Flow	✔	–	–	–	–	✔
Constructability Requirements:							
Tensile Strength	Grab Strength	✔	✔	✔	✔	✔	✔
Seam Strength	Grab Strength	✔	✔	✔	–	✔	–
Bursting Resistance	Burst Strength	✔	✔	✔	✔	✔	✔
Puncture Resistance	Rod or Pyramid Puncture	✔	✔	✔	✔	✔	✔
Tear Resistance	Trapezoidal Tear	✔	✔	✔	✔	✔	✔
Longevity (Durability):							
Abrasion Resistance[3]	Reciprocating Block Abrasion	✔	–	–	–	–	–
UV Stability[4]	UV Resistance	✔	–	–	✔	✔	✔
Soil Environment[5]	Chemical	✔	✔	?	✔	✔	?
	Biological	✔	✔	?	✔	✔	?
	Wet-Dry	✔	✔	–	–	–	✔
	Freeze-Thaw	✔	✔	–	–	✔	–

NOTES
1. See Table 1-3 for specific procedures.
2. Compression creep is applicable to some geocomposites.
3. Erosion control applications where armor stone may move.
4. Exposed geosynthetics only.
5. Where required.

TABLE 1-3
GEOSYNTHETIC PROPERTIES AND PARAMETERS

PROPERTY	TEST METHOD	UNITS OF MEASUREMENT
I. GENERAL PROPERTIES (from manufacturers)		
Type and Construction	N/A	-----
Polymer	N/A	-----
Mass per Unit Area	ASTM D 5261	g/m^2
Thickness (geotextiles & geomembranes)	ASTM D 5199	mm
Roll Length	Measure	m
Roll Widths	Measure	m
Roll Weight	Measure	kg
Roll Diameter	Measure	m
Specific Gravity & Density	ASTM D 792 and D 1505	g/m^3
Surface Characteristics	N/A	-----
II. INDEX PROPERTIES		
<u>MECHANICAL STRENGTH - UNIAXIAL LOADING</u>		
a) Tensile Strength (Quality Control)		
1) Grab Strength (geotextiles & CSPE reinforced geomembranes)	ASTM D 4632	N
2) Single Rib Strength (geogrids)	GRI:GG1	N
3) Narrow Strip (geomembranes)		
- EDPM, CO, IR, CR	ASTM D 412	N
- HDPE	ASTM D 638	N
- PVC, VLDPE	ASTM D 882	N
b) Tensile Strength (Load-Strain Characteristics)		
1) Wide Strip (geotextiles)	ASTM D 4595	N
2) Wide Strip (geogrid)	no standard	N
3) Wide Strip Strength (geomembranes)	ASTM D 4885	N
4) 2% Secant Modulus (PE geomembranes)	ASTM D 882	N
c) Junction Strength (geogrids)	GRI:GG2	%
d) Dynamic Loading	no standard	
e) Creep Resistance	ASTM D 5262 (Note: interpretation required)	creep strain: $\%\epsilon$/hr creep rupture: kN/m
f) Index Friction	GRI:GS7	dimensionless
g) Seam Strength		
1) Sewn (geotextiles)	ASTM D 4884	% efficiency
2) Factory Peel and Shear (geomembranes)	ASTM D 4545	kg/mm
3) Field Peel and Shear (geomembranes)	ASTM D 4437	kg/mm
h) Tear Strength		
1) Trapezoid Tearing (geotextile)	ASTM D 4533	N
2) Tear Resistance (geomembranes)	ASTM D 1004	N
<u>MECHANICAL STRENGTH - RUPTURE RESISTANCE</u>		
a) Burst Strength		
1) Mullen Burst (geotextiles)	ASTM D 3786	Pa
2) CBR (geotextiles, geonets, geomembranes)	GRI:GS1	Pa or N
3) Large Scale Hydrostatic (geomembranes and geotextiles)	ASTM D 5514	Pa
b) Puncture Resistance		
1) Index (geotextiles and geomembranes)	ASTM D 4833	N
2) Pyramid Puncture (geomembranes)	ASTM D 5494	N
3) CBR (geotextile, geonets, and geomembranes)	GRI:GS1	N
c) Penetration Resistance (Dimensional Stability)	no standard	

TABLE 1-3 GEOSYNTHETIC PROPERTIES AND PARAMETERS (continued)

PROPERTY	TEST METHOD	UNITS
II. INDEX PROPERTIES (continued)		
<u>MECHANICAL STRENGTH - RUPTURE RESISTANCE</u> (cont.)		
d) Geosynthetic Cutting Resistance	no standard	
e) Flexibility (Stiffness)	ASTM D 1388	mg/cm^2
<u>ENDURANCE PROPERTIES</u>		
a) Abrasion Resistance (geotextile)	ASTM D 4886	%
b) Ultraviolet (UV) Radiation Stability		
1) Xenon-Arc Apparatus (geotextile)	ASTM D 4355	%
2) Outdoor Exposure	ASTM D 5970	%
c) Chemical Resistance		
1) Chemical Immersion	ASTM D 5322	N/A
2) Oxidative Induction Time	ASTM D 5885	minutes
3) Environmental Exposure	EPA 9090	% change
d) Biological Resistance		
1) Biological Clogging (geotextile)	ASTM D 1987	m^3/s
2) Biological Degradation	ASTM G 21 and G 22	
3) Soil Burial	ASTM D 3083	% change
e) Wet and Dry Stability	no standard	
f) Temperature Stability		
1) Temperature Stability (geotextile)	ASTM D 4594	% change
2) Dimensional Stability (geomembrane)	ASTM D 1204	% change
<u>HYDRAULIC</u>		
a) Opening Characteristics (geotextiles)		
1) Apparent Opening Size (AOS)	ASTM D 4751	mm
2) Porimetry (pore size distribution)	Use AOS for O_{95}, O_{85}, O_{50}, O_{15}, and O_5	mm
3) Percent Open Area (POA)	(see Christopher & Holtz, 1985)	%
4) Porosity (n)	(V_{voids}/V_{total}) 100	%
b) Permeability (k) and Permittivity (Ψ)	ASTM D 4491	m/s and s^{-1}
c) Soil Retention Ability	Empirically Related to Opening Characteristics	
d) Clogging Resistance	ASTM D 5101 and GRI:GT8	
e) In-Plane Flow Capacity (Transmissivity, θ)	ASTM D 4716	m^2/s
III. PERFORMANCE PROPERTIES		
<u>Stress-Strain Characteristics:</u>		kN/m and % strain
a) Tension Test in Soil	(see McGown, et al., 1982)	
b) Triaxial Test Method	(see Holtz, et al., 1982)	
c) CBR on Soil Fabric System	(see Christopher & Holtz, 1985)	
d) Tension Test in Shear Box	(see Christopher & Holtz, 1985)	
<u>Creep Tests:</u>		kN/m and % strain
a) Extension Test in Soil	(see McGown, et al., 1982)	
b) Triaxial Test Method	(see Holtz, et al., 1982)	
c) Extension Test in Shear Box	(see Christopher & Holtz, 1985)	
d) Pullout Method	(see Christopher, et al., 1990)	

TABLE 1-3 GEOSYNTHETIC PROPERTIES AND PARAMETERS (continued)

PROPERTY	TEST METHOD	UNITS
III. PERFORMANCE PROPERTIES (cont.)		
Friction/Adhesion:		
a) Direct Shear (soil-geosynthetic)	ASTM D 5321	degrees (°)
b) Direct Shear (geosynthetic-geosynthetic)	ASTM D 5321	degrees (°)
c) Pullout (geogrids)	GRI:GG5	dimensionless
d) Pullout (geotextiles)	GRI:GT6	dimensionless
e) Anchorage Embedment (geomembranes)	GRI:GM2	kN/m
Dynamic and Cyclic Loading Resistance:	no standard procedures	N/A
Puncture		
a) Gravel, truncated cone or pyramid	ASTM D 5494	kPa
Chemical Resistance:		
a) In Situ Immersion Testing	ASTM D 5496	N/A
Soil Retention and Filtration Properties:		
a) Gradient Ration Method - for noncohesive sand and silt type soils	ASTM D 5101	dimensionless
b) Hydraulic Conductivity Ratio (HCR) - for fine-grained soils	ASTM D 5567	dimensionless
c) Slurry Method - for silt fence applications	ASTM D 5141	%

International (IFAI), is formatted as a Specifier's Guide. General and some index properties are listed according to product type and manufacturer. The Specifier's Guide also contains a directory of product manufacturers, product distributors, geosynthetic installers, design engineers and testing laboratories, with contact person, address, telephone and facsimile numbers noted.

The tests listed in Table 1-3 include *index* tests and *performance* tests. Index tests do not produce an actual design property in most cases, but they do provide a general value from which the property of interest can be qualitatively assessed. Index tests are primarily used by manufacturers for quality control purposes. When determined using identical test procedures, index tests can be used for product comparison, specifications, and quality control evaluation.

On the other hand, performance tests require testing of geosynthetic with its companion material (*e.g.*, soil) to obtain a direct assessment of the property of interest. **Since performance tests should be performed under specific design conditions with soils from the site, manufacturers should no be expected to have the capability or the responsibility to perform such tests.** These tests should be performed under the direction of the design engineer. Performance tests are not normally used in specifications; rather, geosynthetics should be preselected for performance testing based on index values, or performance test results should be correlated to index values for use in specifications.

Brief descriptions of some of the basic properties of geosynthetics (after Christopher and Dahlstrand, 1989) are presented below.

Mass per Unit Area: The unit weight of a geosynthetic is measured in terms of area as opposed to volume due to variations in thickness under normal stress. This property is mainly used to identify materials.

Thickness: Thickness is not usually required information for geotextiles except in permeability-flow calculations. It is used as a primary identifier for geomembranes. When needed, it can be simply obtained using the procedure in Table 1-3, but it must be measured under a specified normal stress. The nominal thickness used for product comparison is measured under a normal stress of 2 kPa for geotextiles and 20 kPa for geogrids and geomembranes.

Tensile Strength: To understand the load-strain characteristics, it is important to consider the complete load-strain curve. It is also important to consider the nature of the test and the testing environment. With most materials, it is usual to use stress in strength and modulus determination. However, because of the thin, two-dimensional nature of geosynthetics, it is awkward to use stress. Therefore, it is conventional with geosynthetics to use force per unit length along the edge of the material. Then, strength and modulus have units of FL^{-1} (*i.e.*, kN/m).

There are several types of tensile strength tests. Specific geosynthetic specimen shapes and loadings are indicated by the referenced procedures in Table 1-3. These tests all give different results.

The plane-strain test represents the loading for many applications, but because it is complicated to perform, it is not a practical test for many routine applications. Therefore, it is approximated by a strip tensile test. Since many narrow strip geosynthetic specimens neck when strained, most applications use wide, short specimens. This is called a wide strip tensile test.

Geosynthetics may have different strengths in different directions. Therefore, tests should be conducted in both principal directions.

The *grab tensile test* is typically used in the specification of geotextiles and is an unusual test. It is widely used and almost universally misused. The grab test may be useful in some applications, but it is difficult, if not impossible, to relate to actual strength without direct correlation tests. The grab tensile test normally uses 25 mm jaws to grip a 100 mm specimen. The strength is reported as the total force needed to cause failure -- not the force per unit width. It is not clear how the force is distributed across the sample. The effects of the specimen being wider than the grips depend on the geotextile filament interaction. In nonwoven geotextiles, these effects are large. In woven geotextiles, they are small.

The *burst test* is performed by applying a normal pressure (usually by air pressure) against a geosynthetic specimen clamped in a ring. The burst strength is given in pascals. This is not the stress in the specimen - it is the normal stress against the geosynthetic at failure. The burst strength depends on the strength in all directions and is controlled by the minimum value. Burst strength is a function of the diameter of the test specimen; therefore, care must be used in comparing tests.

Creep is a time-dependent mechanical property. It is strain at constant load. Creep tests can be run for any of the tensile test types, but are most frequently performed on a wide strip specimen by applying a constant load for a sustained period. Creep tests are influenced by the same factors as tensile load-strain tests - specimen length to width ratio, temperature, moisture, lateral restraint, and confinement.

Short-term creep strain is strongly influenced by the geosynthetic structure. Geogrids and woven geotextiles have the least; heat-bonded geotextiles have intermediate; and needled geotextiles have the most. Longer-term creep rates are controlled by structure and polymer type. Of the most common polymers, polyester has lower creep rates than polypropylene. The creep limit is the most important creep characteristic. It is the load per unit width above which the geosynthetic will creep to rupture. The creep limit is controlled by the polymer and ranges from 20% to 60% of the material's ultimate strength.

<u>Friction</u>: Soil-geosynthetic and geosynthetic-geosynthetic friction are important properties. It is common to assume a soil-geotextile friction value between ⅔ and 1 of the soil angle of friction. For geogrid materials, the value approaches the full friction angle. Caution is advised for geomembranes where soil-geosynthetic friction angle may be much lower than the soil angle of friction. For important applications, tests are justified.

The direct friction test is simple in principle, but numerous details must be considered for accurate results. Recent procedures proposed by ASTM indicate a minimum shear box size of 300 mm by 300 mm to reduce boundary effects. For many geosynthetics, the friction angle is a function of the soils on each side of the geosynthetic and the normal stress; therefore, test conditions must model the actual field conditions.

<u>Durability Properties</u>: Other properties that require consideration are related to durability and longevity. Exposure to ultraviolet light can degrade some geosynthetic properties. The geosynthetic polymer must be compatible with the environment chemistry. The environment should be checked for such items as high and low pH, chlorides, organics and oxidation agents such as ferroginous soils which contain Fe_2SO_3, calcareous soils, and acid sulfate soils that may deteriorate of the geosynthetic in time. Other possible detrimental environmental factors include

chemical solvents, diesel, and other fuels. Each geosynthetic is different in its resistance to aging and attack by chemical and biological agents. Therefore, each product must be investigated individually to determine the effects of these durability factors. The geosynthetic manufacturer should supply the results of product exposure studies, including, but not limited to, strength reduction due to aging, deterioration in ultraviolet light, chemical attack, microbiological attack, environmental stress cracking, hydrolysis, and any possible synergism between individual factors.

Guidelines on soil environments and on geosynthetics properties are presented in the FHWA Corrosion/Degradation of Soil Reinforcements for Mechanically Stabilized Earth Walls and Reinforced Soil Slopes (Elias, 1997). This research has been summarized and numeric recommendations for selecting aging reduction factors for reinforcement applications is presented in an FHWA Geotechnology Technical Note (1997); attached in Appendix F. A durability reduction factor as low as 1.1 is recommended with supporting data.

<u>Hydraulic Properties</u>: Hydraulic properties relate to the pore size distribution of the geosynthetic and correspondingly its ability to retain soil particles over the life of the project while allowing water to pass. Hydraulic properties may also be affected by chemical and biological agents. Ionic deposits as well as slime growth have been known to clog filter systems (granular filters as well as geotextiles).

The ability of a geotextile to retain soil particles is directly related to its apparent opening size (AOS) which is the apparent largest hole in the geotextile. The AOS value is equal to the size of the largest particle that can effectively pass through the geotextile in a dry sieving test.

The ability of water to pass through a geotextile is determined from its hydraulic conductivity (coefficient of permeability, k), as measured in a permeability test. The flow capacity of the material can then be determined from Darcy's law. Due to the compressibility of geotextiles, the permittivity, ψ (permeability divided by thickness), is often determined from the test and used to directly evaluate flow capacity.

The ability of water to pass through a geotextile over the life of the project is dependent on its filtration potential or its ability not to clog with soil particles. Essentially, if the finer particles of soil can pass through the geotextile, it should not clog. Effective filtration can be evaluated through relations between the geotextile's pore size distribution and the soil's grain size distribution; however, such formulations are still in the development phase. For a precise evaluation, laboratory performance testing of the proposed soil and candidate geotextile should be conducted.

One popular filtration test is the gradient ratio test (ASTM D 5101). This test is primarily suitable for sandy and silty soils ($k \leq 10^{-7}$ m/s). In this test, a rigid wall permeameter, with strategically located piezometer ports, is used to obtain a ratio of the head loss in the soil to the head loss at the soil-geotextile interface under different hydraulic gradients. Although the procedure indicates that the test may be terminated after 24 hours, to obtain meaningful results, the test should be continued until stabilization of the flow has clearly occurred. This may occur within 24 hours, but could require several weeks. A gradient ratio of 1 or less is preferred. Less than 1 is an indication that a more open *filter bridge* has developed in the soil adjacent to the geotextile. However, a continued decrease in the gradient ratio indicates piping, and an alternate geotextile should be evaluated. A high gradient ratio indicates a flow reduction at the geotextile. If the gradient ratio approaches 3 (the recommended maximum by the U.S. Army Corps of Engineers, 1977), the flow rate through the system should be carefully evaluated with respect to design requirements. A continued increase in the gradient ratio indicates clogging, and the geotextile is unacceptable.

For fine-grained soils, the hydraulic conductivity ratio (HCR) test (ASTM D 5567) should be considered. This test uses a flexible wall permeameter and evaluates the long-term permeability under increasing gradients with respect to the short-term permeability of the system at the lowest hydraulic gradient. A decrease in HCR indicates a flow reduction in the system. Since measurements are not taken near the geotextile-soil interface and soil permeability is not measured, it is questionable whether an HCR decrease is the result of flow reduction at the geotextile or blinding within the soil matrix itself. An improvement to this method would be to include piezometer or transducers within these zones (after the gradient ratio method) to aid in interpretation of the results.

Other filtration tests for clogging potential include the Caltrans slurry filtration test (Hoover, 1982), which was developed by Legge (1990) into the Fine Fraction filtration (F^3) test (Sansone and Koerner, 1992), and the Long-Term Flow (LTF) test (Koerner and Ko, 1982; GRI Test Method GT1). According to Fischer (1994), all of these tests have serious disadvantages that make them less suitable than the Gradient Ratio (GR) test for determining the filtration behavior of the soil-geotextile system. The GR test must be run longer than the ASTM-specified 24 hours, and proper attention must be paid to the test details (Maré, 1994) to get reproducible results.

Some additional hydraulic properties often required in filtration design are the Percent Open Area (POA) and the porosity. As noted in Table 1-3, there are no standard tests for these properties, although there is a suggested procedure for POA given by Christopher and Holtz (1985), which follows Corps of Engineers procedures. Basically, POA is determined on a light table or by projection enlargement. Porosity is readily calculated just as it is with soils; that is, porosity is the volume of the voids divided by the total volume. The total volume is, for example, 1 m^2, times the nominal thickness of the geotextile. The volume of voids is the total volume minus the volume of the fibers and filaments (*solids*), or the mass of 1 m^2 divided by the specific gravity of the polymer.

1.6 SPECIFICATIONS

Specifications should be based on the specific geosynthetic properties required for design and installation. *Standard* geosynthetics may result in uneconomical or unsafe designs. To specify a particular type of geosynthetic or its *equivalent* can also be very misleading. As a result, the contractor may select a product that has completely different properties than intended by the designer. In almost every chapter of this manual, guide specifications are given for the particular application discussed in the chapter. See Richardson and Koerner (1990) and Koerner and Wayne (1989) for additional guide specifications.

All geosynthetic specifications should include:

- general requirements
- specific geosynthetic properties
- seams and overlaps
- placement procedures
- repairs, and
- acceptance and rejection criteria

General requirements include the types of geosynthetics, acceptable polymeric materials, and comments related to the stability of the material. Geosynthetic manufacturers and representatives are good sources of information on these characteristics. Other items that should be specified in this section are instructions on storage and handling so products can be protected from ultraviolet exposure, dust, mud, or any other elements that may affect performance. Guidelines concerning on-site storage and handling of geotextiles are contained in ASTM D 4873, Standard Guide for Identification, Storage, and Handling of Geotextiles. If pertinent, roll weight and dimensions may also be specified. Finally, certification requirements should be included in this section.

Specific geosynthetic physical, index, and performance properties as required by the design must be listed. Properties should be given in terms of *minimum (or maximum) average roll values* (MARVs), along with the required test methods. MARVs are simply the smallest (or largest) anticipated *average* value that would be obtained for any roll tested (Koerner, 1994). This *average* property value must exceed the minimum (or be less than the maximum) value specified for that property based on a particular test. Ordinarily it is possible to obtain a manufacturer's certification for MARVs.

If performance tests have been conducted as part of the design, a list of approved products could be provided. **The language *or equal* and *or equivalent* should be avoided within the specification, unless equivalency is spelled out in terms of the index properties and the performance criteria that were required to be included on the approved list.** Approved lists can also be developed based on experience with recurring application conditions. Once an approved list has been established, new geosynthetics can be added as they are approved. Manufacturer's samples should be periodically obtained so they can be examined alongside the original tested specimens to verify whether the manufacturing process has changed since the product was approved. Development of an approved list program will take considerable initial effort, but once established, it provides a simple, convenient method of specifying geosynthetics with confidence.

Seam and overlap requirements should be specified along with the design properties for both factory and field seams, as applicable. A minimum overlap of 0.3 m is recommended for all geotextile applications, but overlaps may be increased due to specific site and construction requirements. Sewing of seams, discussed in Section 1.8, may be required for special conditions. Also, certain geotextiles may have factory seams. The seam strengths specified should equal the required strength of the geosynthetic, in the direction perpendicular to the seam length, using the same test procedures. For designs where wide width tests are used (*e.g.*, reinforced embankments on soft foundations), the required seam strength is a calculated design value. Therefore, seam strengths should not be specified as a percent of the geosynthetic strength.

Geogrids and geonets may be connected by mechanical fasteners, though the connection may be either structural or a construction aid (*i.e.*, strength perpendicular to the seam length is not required by design). Geomembranes are normally thermally bonded and specified in terms of peel and shear seam strengths, as discussed in Chapter 10.

For sewn geotextiles, geomembranes, and structurally connected geogrids, the seaming material (thread, extrudate, or fastener) should consist of polymeric materials that have the same or greater durability as the geosynthetic being seamed. For example, nylon thread, unless treated, which is often used for geotextile seams may weaken in time as it absorbs water.

Placement procedures should be given in detail within the specification and on the construction drawings. These procedures should include grading and ground-clearing requirements, aggregate specifications, aggregate lift thickness, and equipment requirements. These requirements are especially important if the geosynthetic was selected on the basis of survivability. Detailed placement procedures are presented in each application chapter.

Repair procedures for damaged sections of geosynthetics (*i.e.*, rips and tears) should be detailed. Such repairs should include requirements for overlaps, sewn seams, fused seams, or replacement requirements. For overlap repairs, the geosynthetic should extend the minimum of the overlap length requirement from all edges of the tear or rip (*i.e.*, if a 0.3 m overlap is required, the patch should extend at least 0.3 m from all edges of the tear).

Acceptance and rejection criteria for the geosynthetic materials should be clearly and concisely stated in the specifications. It is very important that all installations be observed by a designer representative who is knowledgeable in geotextile placement procedures and who is aware of design requirements. Sampling (*e.g.*, ASTM D 4354, Standard Practice for Sampling of Geosynthetics for Testing) and testing requirements be required during construction should also be specified. Guidelines for acceptance and rejection of geosynthetic shipments are contained in ASTM D 4759, Standard Practice for Determining the Specification Conformance of Geosynthetics.

For small projects, the cost of ASTM acceptance/rejection criterion testing is often a significant portion of the total project cost and may even exceed the cost of the geosynthetic itself. In such cases, a manufacturer's product certification specification requirement or an approved product list type specification may be satisfactory.

1.7 SPECIFICATION CONFORMANCE EXAMPLE

<u>DEFINITION OF EXAMPLE</u>

- Project Description: a geotextile separator will be used in construction of a roadway

<u>GIVEN DATA</u>

- a Class 2 (AASHTO M 288) geotextile was specified for survivability
- 110 rolls of geotextile are required for the project, and have arrived on site in one shipment
- geotextile is a nonwoven, with an elongation at failure (per ASTM D 4632) of greater than 50%
- test results for the samples are presented in the table below
- the coefficient of variation for the test laboratory is undefined

DETERMINE

- whether the geotextile meets the required grab tensile strength of 700 N

SOLUTION

A. What is the lot size?

The lot size is 110, the number of rolls shipped to this project.

B. How many units, or number of rolls, should be selected for as samples for conformance testing?

The total number of units, or rolls, in this lot is 110. The number of rolls to take lot samples from is 5, per ASTM D 4354, Standard Practice for Sampling of Geosynthetics for Testing.

C. How many sampling units should be take from each roll?

One laboratory sampling unit should be taken from each roll (lot sampling unit), per ASTM D 4632, Standard Test Method for Grab Breaking Load and Elongation of Geotextiles.

D. How many test specimens per laboratory sampling unit, in each direction, are required?

Since the coefficient of variation is undefined for the test laboratory (in this example), specify the fixed number of 10 specimens per roll, in both the machine and cross-machine directions, are required. This is based upon an assumed v = 9.5%, which is somewhat larger than usually found in practice.

The number of tests may be reduced, with the following equation, when the laboratory's coefficient of variation is defined. Test Method D 4632 defines the following number of test specimens per laboratory sampling unit in each direction:

$$n = (tv/A)^2$$

where:

n = number of test specimens per laboratory sampling unit (rounded upward to the next whole number);

v = reliable estimate of the coefficient of variation for individual observations based on similar materials in the user's laboratory under conditions of single-operator precision, %;

t = the value of Student's t for one-sided limits, a 95% probability level, and the degrees of freedom associated with the estimate of v; and

A = 5.0% of the average, the value of allowable variation.

Per Test Method D 4632 if there is no reliable estimate of v for the user's laboratory, the equation above should not be used directly. Instead, specify the fixed number of 10 specimens for the machine direction tests and 10 specimens for the cross-machine direction test.

Test Results — Machine Direction

	Roll Number				
Specimen	1	2	3	4	5
1	720	733	687	702	693
2	713	715	715	689	701
3	715	721	717	707	698
4	708	719	706	716	711
5	707	707	724	730	707
6	700	713	699	724	720
7	699	720	705	717	725
8	711	703	712	712	720
9	717	700	717	707	718
10	703	712	722	716	715
Average	709.3	714.3	710.4	712.0	710.8

Test Results — Cross-Machine Direction

	Roll Number				
Specimen	1	2	3	4	5
1	715	723	683	699	690
2	708	710	710	687	700
3	710	711	707	701	691
4	703	717	707	706	701
5	709	709	713	723	706
6	703	715	688	719	718
7	689	710	701	707	721
8	701	708	700	702	719
9	707	707	693	707	713
10	700	710	701	710	711
Average	704.5	712.0	700.3	706.1	707.0

All roll averages exceed the specification value of 700 N. Therefore, the grab strength of this lot is acceptable.

1.8 FIELD INSPECTION

Problems with geosynthetic applications are often attributed to poor product acceptance and construction monitoring procedures on the part of the owner, and/or inappropriate installation methods on the part of the contractor. A checklist for field personnel responsible for observing a geosynthetic installation is presented in Table 1-4. Recommended installation methods are presented in the application chapters.

1.9 FIELD SEAMING

Some form of geosynthetic seaming will be utilized in those applications that require continuity between adjacent rolls. Seaming techniques include overlapping, sewing, stapling, tying, heat bonding, welding and gluing. Some of these techniques are more suitable for certain types of geosynthetics than others. For example, the most efficient and widely used methods for geotextiles are overlapping and sewing, and these techniques are discussed first.

The first technique, the *simple overlap*, will be suitable for most geotextile and biaxial geogrid projects. The minimum overlap is 0.3 m. Greater overlaps are required for specific applications. If stress transfer is required between adjacent rolls, the only strength provided by an overlap is the friction between adjacent sheets of geotextiles, and by friction and fill *strike-through* of substantial apertures of biaxial geogrids. Unless overburden pressures are large and the overlap substantial, very little stress can actually be transferred through the overlap.

The second technique, *sewing*, offers a practical and economical alternative for geotextiles when overlaps become excessive or stress transfer is required between two adjacent rolls of fabric. For typical projects and conditions, sewing is generally more economical when overlaps of 1 m or greater are required. To obtain good-quality, effective seams, the user should be aware of the following sewing variables (Koerner, 1994; Diaz and Myles, 1990; Ko, 1987):

- Thread type: Kevlar aramid, polyethylene, polyester, or polypropylene (in approximate order of decreasing strength and cost). Thread durability must be consistent with project requirements.
- Thread tension: Usually adjusted in the field to be sufficiently tight; but not cut the geotextile.
- Stitch density: Typically, 200 to 400 stitches per meter are used for lighter-weight geotextiles, while heavier geotextiles usually allow only 150 to 200 stitches per meter.
- Stitch type: Single- or double-thread chainstitch, Types 101 or 401; with double-thread chain- or *lock*-stitch preferred because it is less likely to unravel (Figure 1-2(a)).
- Number of rows: Usually two or more parallel rows are preferred for increased safety.
- Seam type: Flat or *prayer* seams, J- or Double J-type seams, or *butterfly* seams are the most widely used (Figure 1-2(b)).

TABLE 1-4

GEOSYNTHETIC FIELD INSPECTION CHECKLIST

☐ 1. Read the specifications; determine if geosynthetic is specified by (a) specific properties or (b) an approved products list.

☐ 2. Review the construction plans.

☐ 3. (a) For specification by specific properties, check listed material properties of supplied geosynthetic, from published literature, against the specific property values specified.

OR

(b) Obtain the geosynthetic name(s), type, and style, along with a small sample(s) of approved material(s) from the design engineer. Check supplied geosynthetic type and style for conformance to approved material(s). If the geosynthetic is not listed, contact the designer with a description of the material and request evaluation and approval or rejection.

☐ 4. On site, check the rolls of geosynthetics to see that they are properly stored; check for any damage.

☐ 5. Check roll and lot numbers to verify whether they match certification documents.

☐ 6. Cut two samples 100 mm to 150 mm square from a roll. Staple one to your copy of the specifications for comparison with future shipments and send one to the design engineer for approval or information.

☐ 7. Observe materials in each roll to make sure they are the same. Observe rolls for flaws and nonuniformity.

☐ 8. Obtain test samples according to specification requirements from randomly selected rolls. Mark the machine direction on each sample and note the roll number.

☐ 9. Observe construction to see that the contractor complies with specification requirements for installation.

☐ 10. Check all seams, both factory and field, for any flaws (*e.g.*, missed stitches in geotextile). If necessary, either reseam or reject materials.

☐ 11. If possible, check geosynthetic after aggregate or riprap placement for possible damage. This can be done either by constructing a trial installation, or by removing a small section of aggregate or riprap and observing the geosynthetic after placement and compaction of the aggregate, at the beginning of the project. If perforations, tears, or other damage has occurred, contact the design engineer.

☐ 12. Check future shipments against the initial approved shipment and collect additional test samples. Collect samples of seams, both factory and field, for testing. For field seams, have the contractor sew several meters of a *dummy* seam(s) for testing and evaluation.

When constructed correctly, sewn seams can provide reliable stress transfer between adjacent sheets of geotextile. However, there are several points with regard to seam strength that should be understood, as follows.

1. Due to needle damage and stress concentrations at the stitch, sewn seams are weaker than the geotextile (good, high-quality seams have only about 50% to 80% of the intact geotextile strength based on wide width tests).
2. Grab strength results are influenced by the stitches, so the test yields artificially high seam strengths. Grab test should only be used for quality control and not to determine strength.
3. The maximum seam strengths achievable at this time are on the order of 200 kN/m under factory conditions, using 330 kN/m geotextiles.
4. Field seam strengths will most likely be lower than laboratory or factory seam strengths.
5. All stitches can unravel, although lock-type stitches are less likely to.
6. Unraveling can be avoided by utilizing high-quality equipment and proper selection of needles, thread, seam and stitch type, and by using two or more rows of stitches.
7. Careful inspection of all stitches is essential.

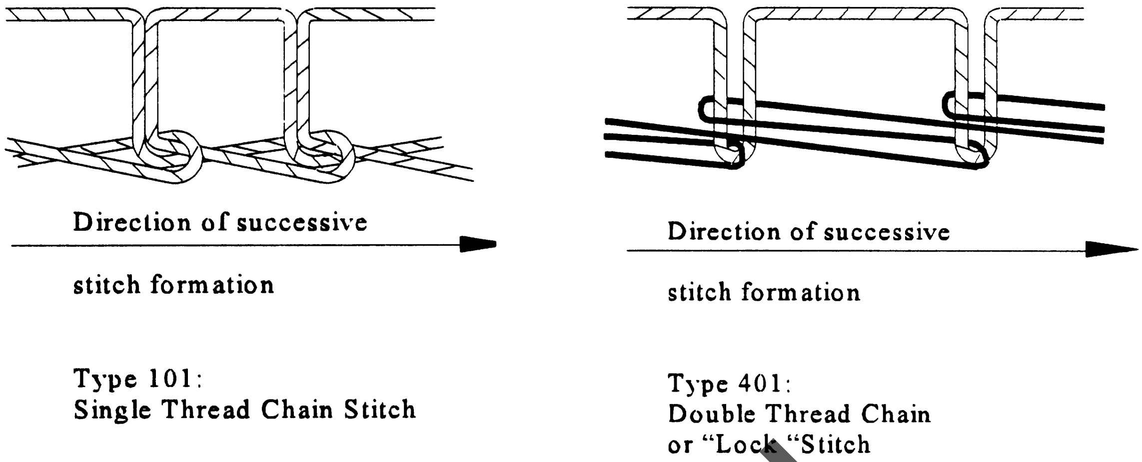
Field sewing is relatively simple and usually requires two or three laborers, depending on the geotextile, seam type, and sewing machine. Good seams require careful control of the operation, cleanliness, and protection from the elements. However, adverse field conditions can easily complicate sewing operations. Although most portable sewing machines are electric, pneumatic equipment is available for operating in wet environments.

Since the seam is the weakest link in the geotextile, all seams, including factory seams, should be carefully inspected. To facilitate inspection and repair, the geotextile should be placed (or at least inspected prior to placement) with all seams up (Figure. 1-2(c)). Using a contrasting thread color can facilitate inspection. Procedures for testing sewn seams are given in ASTM D 4884, Standard Test Method for Seam Strength of Sewn Geotextiles.

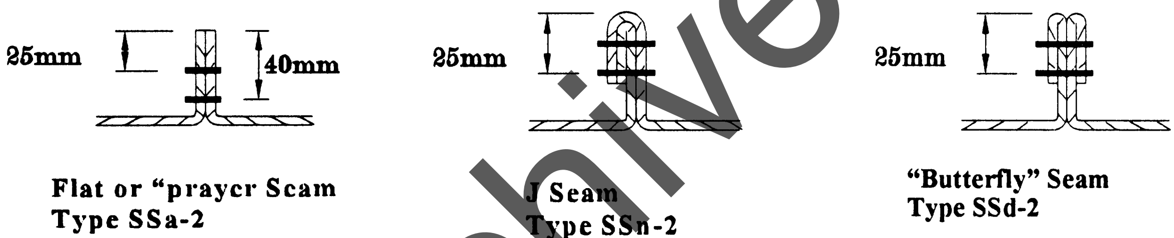
Seaming of biaxial geogrids and geocomposites is most commonly achieved by overlaps, and the remarks above on overlap of geotextiles are generally appropriate to these products. Uniaxial geogrids are normally butted in the along-the-roll direction. Seams in the roll direction of uniaxial geogrids are made with a *bodkin* joint for HDPE geogrids, as illustrated in Figure 1-3, and may be made with overlaps for coated PET geogrids.

Seaming of geomembranes and other geosynthetic barriers is much more varied. The method of seaming is dependent upon the geosynthetic material being used and the project design. Overlaps of a designated length are typically used for thin-film geotextile composites and geosynthetic clay liners. Geomembranes are seamed with thermal methods or with solvents.

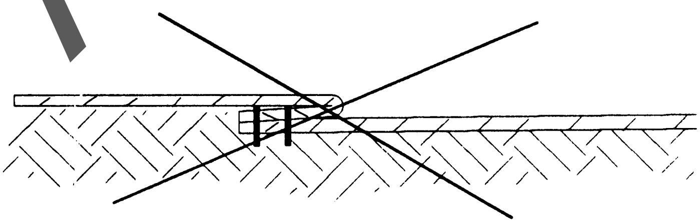
(a) Type of stitches



(b) Type of seams



(c) Improper placement



Cannot Inspect or Repair

Figure 1-2 Types of (a) stitches and (b) seams, according to Federal Standard No. 751a (1965); and (c) improper seam placement.

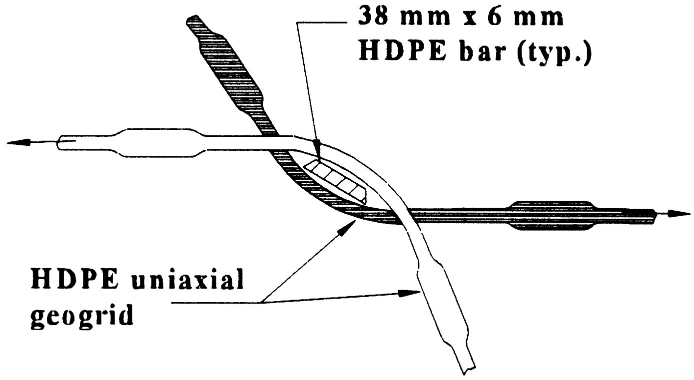


Figure 1-3 Bodkin connection of HDPE uniaxial geogrid.

1.10 REFERENCES

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2.0 GEOSYNTHETICS IN SUBSURFACE DRAINAGE SYSTEMS

2.1 BACKGROUND

One major area of geotextile use is as filters in drain applications such as trench and interception drains, blanket drains, pavement edge drains, structure drains, and beneath permeable roadway bases. The *filter* restricts movement of soil particles as water flows into the *drain* structure and is collected and/or transported downstream. Geocomposites consisting of a drainage core surrounded by a geotextile filter are often used as the drain itself in these applications. Geotextiles are also used as filters beneath hard armor erosion control systems, and this application will be discussed in Chapter 3.

Because of their comparable performance, improved economy, consistent properties, and ease of placement, geotextiles have been used successfully to replace graded granular filters in almost all drainage applications. Thus, they must perform the same functions as graded granular filters:

- to allow water to flow through the filter into the drain, and to continue doing this throughout the life of the project; and
- to retain the soil particles in place and prevent their migration (*piping*) through the filter (if some soil particles do move, they must be able to pass through the filter without blinding or clogging the downstream media during the life of the project).

Geotextiles, like graded granular filters, require proper engineering design or they may not perform as desired. Unless flow requirements, piping resistance, clogging resistance and constructability requirements (defined later) are properly specified, the geotextile/soil filtration system may not perform properly. In addition, construction must be monitored to ensure that materials are installed correctly.

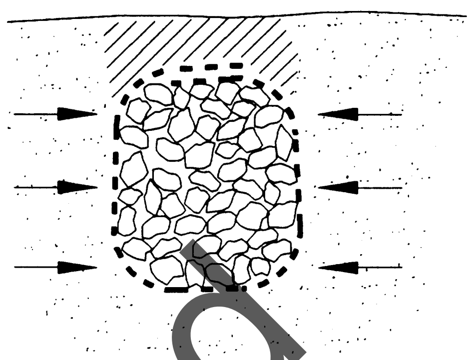
In most drainage and filtration applications, geotextile use can be justified over conventional graded granular filter material use because of cost advantages from:

- the use of less-costly drainage aggregate;
- the possible use of smaller-sized drains;
- the possible elimination of collector pipes;
- expedient construction;
- lower risk of contamination and segregation of drainage aggregate during construction;
- reduced excavation.

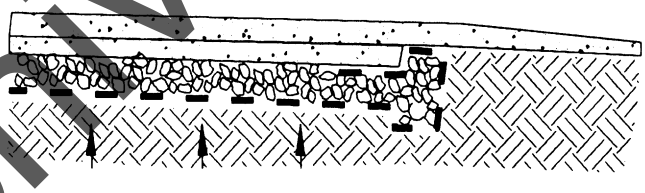
2.2 APPLICATIONS

Properly designed geotextiles can be used as a replacement for, or in conjunction with, conventional graded granular filters in almost any drainage application. Properly designed geocomposites can be used as a replacement for granular drains in many applications (*e.g.*, pavement edge drains). Below are a few examples of drainage applications.

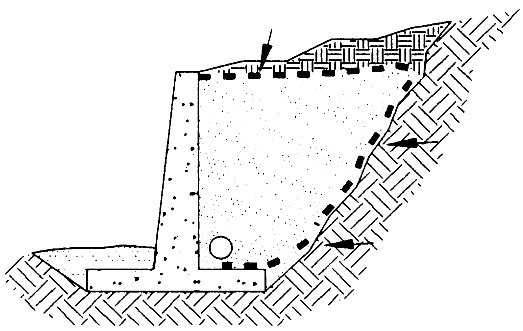
- Filters around trench drains and edge drains -- to prevent soil from migrating into the drainage aggregate or system, while allowing water to exit from the soil.



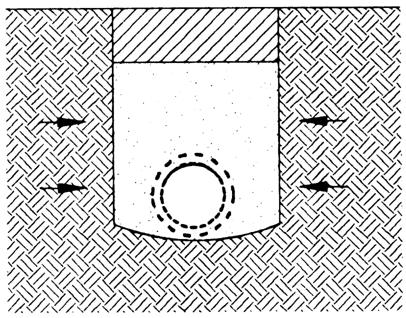
- Filters beneath pavement permeable bases, blanket drains and base courses. Prefabricated geocomposite drains and geotextile-wrapped trenches are used in pavement edge drain construction.



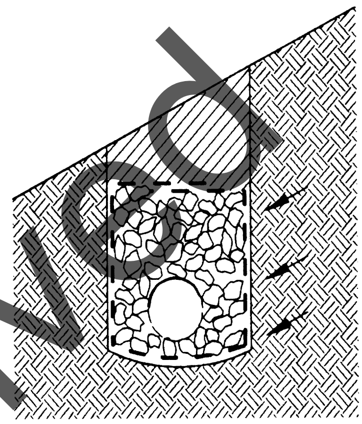
- Drains for structures such as retaining walls and bridge abutments. They separate the drainage aggregate or system from the backfill soil, while allowing free drainage of ground and infiltration water. Geocomposite drains are especially useful in this application.



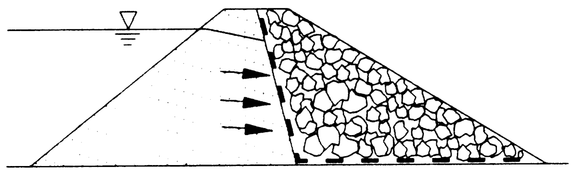
- Geotextile wraps for slotted or jointed drain and well pipes -- to prevent filter aggregate from entering the pipe, while allowing the free flow of water into the pipe.



- Interceptor, toe drains, and surface drains -- to aid in the stabilization of slopes by allowing excess pore pressures within the slope to dissipate, and by preventing surface erosion. Again, geocomposites have been successfully used in this application.



- Chimney and toe drains for earth dams and levees -- to provide seepage control.



In each of these applications, flow is through the geotextile -- that is, perpendicular to the plane of the fabric. In other applications, such as vertical drains in soft foundation soils, lateral drains below slabs and behind retaining walls, and gas transfer media, flow may occur both perpendicular to and transversely in the plane of the geotextile. In many of these applications, geocomposite drains may be appropriate. Design with geocomposite systems is covered in Section 2.11.

All geosynthetic designs should begin with a criticality and severity assessment of the project conditions (see Table 2-1) for a particular application. Although first developed by Carroll (1983) for drainage and filtration applications, the concept of critical-severe projects -- and, thus, the level of engineering responsibility required -- will be applied to other geosynthetic applications throughout this manual.

TABLE 2-1
GUIDELINES FOR EVALUATING THE CRITICAL NATURE OR SEVERITY OF DRAINAGE AND EROSION CONTROL APPLICATIONS
(after Carroll, 1983)

A. Critical Nature of the Project		
Item	Critical	Less Critical
1. Risk of loss of life and/or structural damage due to drain failure:	High	None
2. Repair costs versus installation costs of drain:	>>>	= or <
3. Evidence of drain clogging before potential catastrophic failure:	None	Yes
B. Severity of the Conditions		
Item	Severe	Less Severe
1. Soil to be drained:	Gap-graded, pipable, or dispersible	Well-graded or uniform
2. Hydraulic gradient:	High	Low
3. Flow conditions:	Dynamic, cyclic, or pulsating	Steady state

A few words about the condition of the soil to be drained (Table 2-1) are in order. First, gap-graded, well-graded and uniform soils are illustrated in Figure 2-1. Certain gap-graded and broadly graded soils may be *internally unstable*; that is, they can experience piping or internal erosion. On the other hand, a soil is *internally stable* if it is self-filtering and if its own fine particles do not move through the pores of its coarser fraction (LaFluer, et al., 1993). Criteria for deciding whether a soil is internally unstable will be given in the next section.

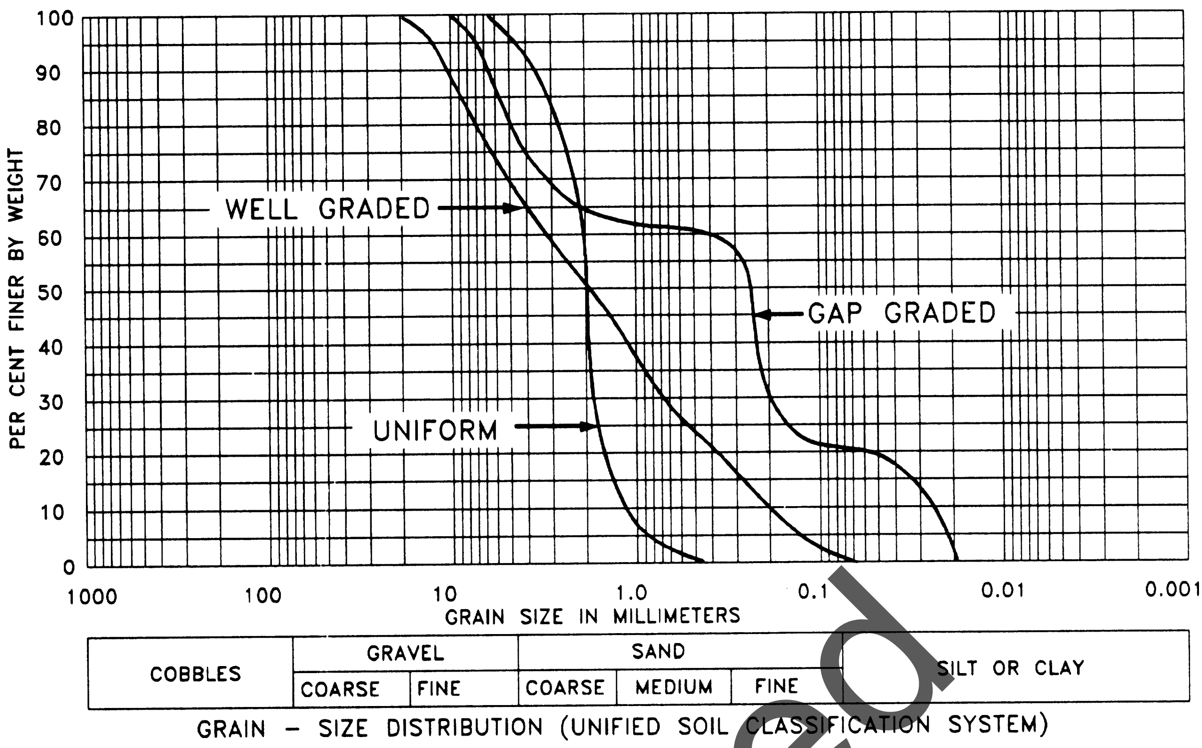


Figure 2-1 Soil descriptions.

Dispersible soils are fine-grained natural soils which deflocculate in the presence of water and, therefore, are highly susceptible to erosion and piping (Sherard, et al., 1972). See also Sherard and Decker (1977) for more information on dispersible soils.

2.3 GEOTEXTILE FILTER DESIGN

Designing with geotextiles for filtration is essentially the same as designing graded granular filters. A geotextile is similar to a soil in that it has voids (pores) and particles (filaments and fibers). However, because of the shape and arrangement of the filaments and the compressibility of the structure with geotextiles, the geometric relationships between filaments and voids is more complex than in soils. In geotextiles, pore size is measured directly, rather than using particle size as an estimate of pore size, as is done with soils. Since pore size can be directly measured, relatively simple relationships between the pore sizes and particle sizes of the soil to be retained can be developed. Three simple filtration concepts are used in the design process:

1. If the size of the largest pore in the geotextile filter is smaller than the larger particles of soil, the soil will be retained by the filter. As with graded granular filters, the larger

particles of soil will form a filter bridge over the hole, which in turn, filters smaller particles of soil, which then retain the soil and prevent piping (Figure 2-2).

2. If the smaller openings in the geotextile are sufficiently large enough to allow smaller particles of soil to pass through the filter, then the geotextile will not *blind* or *clog* (see Figure 2-3).
3. A large number of openings should be present in the geotextile so proper flow can be maintained even if some of the openings later become plugged.

These simple concepts and analogies with soil filter design criteria are used to establish design criteria for geotextiles. Specifically, these criteria state:

- the geotextile must retain the soil (retention criterion), while
- allowing water to pass (permeability criterion), throughout
- the life of the structure (clogging resistance criterion).

To perform effectively, the geotextile must also survive the installation process (survivability criterion).

After a detailed study of research carried out both in North America and in Europe on conventional and geotextile filters, Christopher and Holtz (1985) developed the following design procedure for geotextile filters for drainage (this chapter) and permanent erosion control applications (Chapter 3). The level of design required depends on the critical nature of the project and the severity of the hydraulic and soil conditions (Table 2-1). Especially for critical projects, consideration of the risks and the consequences of geotextile filter failure require great care in selecting the appropriate geotextile. For such projects, and for severe hydraulic conditions, conservative designs are recommended. **Geotextile selection should not be based on cost alone.** The cost of the geotextile is usually minor in comparison to the other components and the construction costs of a drainage system. Also, do not try to save money by eliminating laboratory soil-geotextile performance testing when such testing is required by the design procedure.

A recent National Cooperative Highway Research Program (NCHRP) study (Koerner et al., 1994) of the performance of geotextile drainage systems indicated that the FHWA design criteria developed by Christopher and Holtz (1985) were an excellent prediction of filter performance, particularly for granular soils ($<50\%$ passing a 0.075 mm sieve).

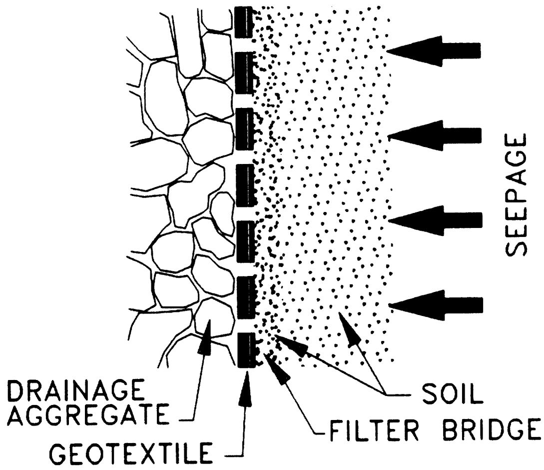


Figure 2-2 Filter bridge formation.

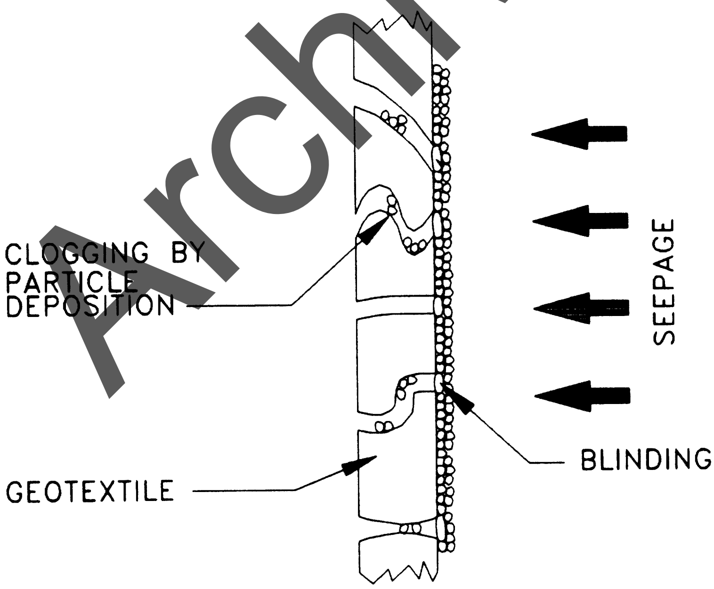


Figure 2-3 Definitions of clogging and blinding (Bell and Hicks, 1980).

2.3-1 Retention Criteria

2.3-1.a Steady State Flow Conditions

$$\text{AOS or } O_{95(\text{geotextile})} \leq B\, D_{85\,(\text{soil})} \qquad [2 - 1]$$

where:

AOS = apparent opening size (see Table 1-3) (mm);
O_{95} = opening size in the geotextile for which 95% are smaller (mm);
AOS ≈ O_{95};
B = a coefficient (dimensionless); and
D_{85} = soil particle size for which 85% are smaller (mm).

The coefficient B ranges from 0.5 to 2 and is a function of the type of soil to be filtered, its density, the uniformity coefficient C_u if the soil is granular, the type of geotextile (woven or nonwoven), and the flow conditions.

For *sands, gravelly sands, silty sands,* and *clayey sands* (with less than 50% passing the 0.075 mm sieve per the Unified Soil Classification System), B is a function of the uniformity coefficient, C_u. Therefore, for

$$C_u \leq 2 \text{ or } \geq 8: \qquad B = 1 \qquad [2 - 2a]$$

$$2 \leq C_u \leq 4: \qquad B = 0.5\, C_u \qquad [2 - 2b]$$

$$4 < C_u < 8 \qquad B = 8/C_u \qquad [2 - 2c]$$

where:

C_u = D_{60}/D_{10}.

Sandy soils which are not uniform (Figure 2-1) tend to bridge across the openings; thus, the larger pores may actually be up to twice as large (B ≤ 2) as the larger soil particles because, quite simply, two particles cannot pass through the same hole at the same time. Therefore, use of the criterion B = 1 would be quite conservative for retention, and such a criterion has been used by, for example, the Corps of Engineers.

If the protected soil contains any fines, use only the portion passing the 4.75 mm sieve for selecting the geotextile (*i.e.*, scalp off the +4.75 mm material).

For *silts and clays* (with more than 50% passing the 0.075 mm sieve), B is a function of the type of geotextile:

for *wovens*, $B = 1;\ O_{95} \leq D_{85}$ [2 - 3]

for *nonwovens*, $B = 1.8;\ O_{95} \leq 1.8\ D_{85}$ [2 - 4]

and *for both*, AOS or $O_{95} \leq 0.3$ mm [2 - 5]

Due to their random pore characteristics and, in some types, their felt-like nature, nonwovens will generally retain finer particles than a woven geotextile of the same AOS. Therefore, the use of $B = 1$ will be even more conservative for nonwovens.

In absence of detailed design, the AASHTO M 288 Standard Specification for Geotextiles (1997) provides the following recommended maximum AOS values in relation to percent of situ soil passing the 0.075 mm sieve: (i) 0.43 mm for less than 15% passing; (ii) 0.25 mm for 15 to 50% passing; and (iii) 0.22 mm for more than 50% passing. However, for cohesive soils with a plasticity index greater than 7, the maximum AOS size is 0.30 mm. These default AOS values are based upon the predominant particle sizes of the in situ soil. The engineer may require performance testing based on engineering design for drainage systems in problematic soil environments. Site specific testing should be performed especially if one or more of the following problematic soil environments are encountered: unstable or highly erodible soils such as non-cohesive silts; gap graded soils; alternating sand/silt laminated soils; dispersive clays; and/or rock flour.

2.3-1.b Dynamic Flow Conditions

If the geotextile is not properly weighted down and in *intimate contact* with the soil to be protected, or if dynamic, cyclic, or pulsating loading conditions produce high localized hydraulic gradients, then soil particles can move behind the geotextile. Thus, the use of $B = 1$ is not conservative, because the bridging network will not develop and the geotextile will be required to retain even finer particles. When retention is the primary criteria, B should be reduced to 0.5; or:

$$O_{95} \leq 0.5\ D_{85} \qquad [2 -6]$$

Dynamic flow conditions can occur in pavement drainage applications. For reversing inflow-outflow or high-gradient situations, it is best to maintain sufficient weight or load on the filter to prevent particle movement. Dynamic flow conditions with erosion control systems are discussed in Chapter 3.

2.3-1.c Stable versus Unstable Soils

The above retention criteria assumes that the soil to be filtered is internally stable -- it will not pipe internally. If unstable soil conditions are encountered, performance tests should be conducted to select suitable geotextiles. According to Kenney and Lau (1985, 1986) and LaFluer, et al. (1989), broadly graded ($C_u > 20$) soils with concave upward grain size distributions tend to be internally unstable. The Kenney and Lau (1985, 1986) procedure utilizes a mass fraction analysis. Research by Skempton and Brogan (1994) verified the Kenney and Lau (1985, 1986) procedure.

2.3-2 Permeability/Permittivity Criteria

Permeability requirements:

-- for *less critical applications* and *less severe conditions*:

$$k_{geotextile} \geq k_{soil} \qquad [2 - 7a]$$

-- and, for *critical applications* and *severe conditions*:

$$k_{geotextile} \geq 10\, k_{soil} \qquad [2 - 7b]$$

Permittivity requirements:

$$\psi \geq 0.5 \text{ sec}^{-1} \text{ for} < 15\% \text{ passing } 0.075 \text{ mm} \qquad [2 - 8a]$$

$$\psi \geq 0.2 \text{ sec}^{-1} \text{ for } 15 \text{ to } 50\% \text{ passing } 0.075 \text{ mm} \qquad [2 - 8b]$$

$$\psi \geq 0.1 \text{ sec}^{-1} \text{ for} > 50\% \text{ passing } 0.075 \text{ mm} \qquad [2 - 8c]$$

In these equations:

k = Darcy coefficient of permeability (m/s); and

ψ = geotextile permittivity, which is equal to $k_{geotextile}/t_{geotextile}$ (1/s) and is a function of the hydraulic head.

For actual flow capacity, the permeability criteria for noncritical applications is conservative, since an equal quantity of flow through a relatively thin geotextile takes significantly less time than through a thick granular filter. Even so, some pores in the geotextile may become blocked or plugged with time. Therefore, for critical or severe applications, Equation 2-7b is recommended to provide an additional level of conservatism. Equation 2-7a may be used where flow reduction is judged not to be a problem, such as in clean, medium to coarse sands and gravels.

The AASHTO M 288 Standard Specification for Geotextiles (1997) presents recommended minimum permittivity values in relation to percent of situ soil passing the 0.075 mm sieve. The values are the same as presented in Equations 2-8a, 2-8b, and 2-8c above. The default permittivity values are based upon the predominant particle sizes of the in situ soil. Again, the engineer may require performance testing based on engineering design for drainage systems in problematic soil environments.

The required flow rate, q, through the system should also be determined, and the geotextile and drainage aggregate selected to provide adequate capacity. As indicated above, flow capacities should not be a problem for most applications, provided the geotextile permeability is greater than the soil permeability. However, in certain situations, such as where geotextiles are used to span joints in rigid structures and where they are used as pipe wraps, portions of the geotextile may be blocked. For these applications, the following criteria should be used together with the permeability criteria:

$$q_{required} = q_{geotextile}(A_g/A_t) \qquad [2 - 9]$$

where:

A_g = geotextile area available for flow; and

A_t = total geotextile area.

2.3-3 Clogging Resistance

2.3-3.a Less Critical/Less Severe Conditions

For *less critical/less severe* conditions:

$$O_{95\ (geotextile)} \geq 3\ D_{15\ (soil)} \qquad [2 - 10]$$

Equation 2-10 applies to soils with $C_u > 3$. For $C_u \leq 3$, select a geotextile with the maximum AOS value from Section 2.3.1.

In situations where clogging is a possibility (*e.g.*, gap-graded or silty soils), the following *optional* qualifiers may be applied:

for *nonwovens* -

$$\text{porosity of the geotextile, } n \geq 50\% \qquad [2 - 11]$$

for *woven monofilament* and *slit film wovens* -

$$\text{percent open area, POA} \geq 4\% \qquad [2 - 12]$$

NOTE: See Section 1.5 for comments on porosity and POA.

Most common nonwovens have porosities much greater than 70%. Most woven monofilaments easily meet the criterion of Equation 2-12; tightly woven slit films do not, and are therefore not recommended for subsurface drainage applications.

Filtration tests provide another option for consideration, especially by *inexperienced users*.

2.3-3.b Critical/Severe Conditions

For *critical/severe* conditions, select geotextiles that meet the retention and permeability criteria in Sections 2.3-1 and 2.3-2. Then perform a filtration test using samples of on-site soils and hydraulic conditions. One type of filtration test is the gradient ratio test (ASTM D 5101).

Although several empirical methods have been proposed to evaluate geotextile filtration characteristics (*i.e.*, the clogging potential), the most realistic approach for all applications is to perform a laboratory test which simulates or models field conditions. We recommend the gradient ratio test, ASTM D 5101, Measuring the Soil-Geotextile System Clogging Potential by the Gradient Ratio. This test utilizes a rigid-wall soil permeameter with piezometer taps that allow for simultaneous measurement of the head losses in the soil and the head loss across the soil/geotextile interface (Figure 2-4). The ratio of the head loss across this interface (nominally

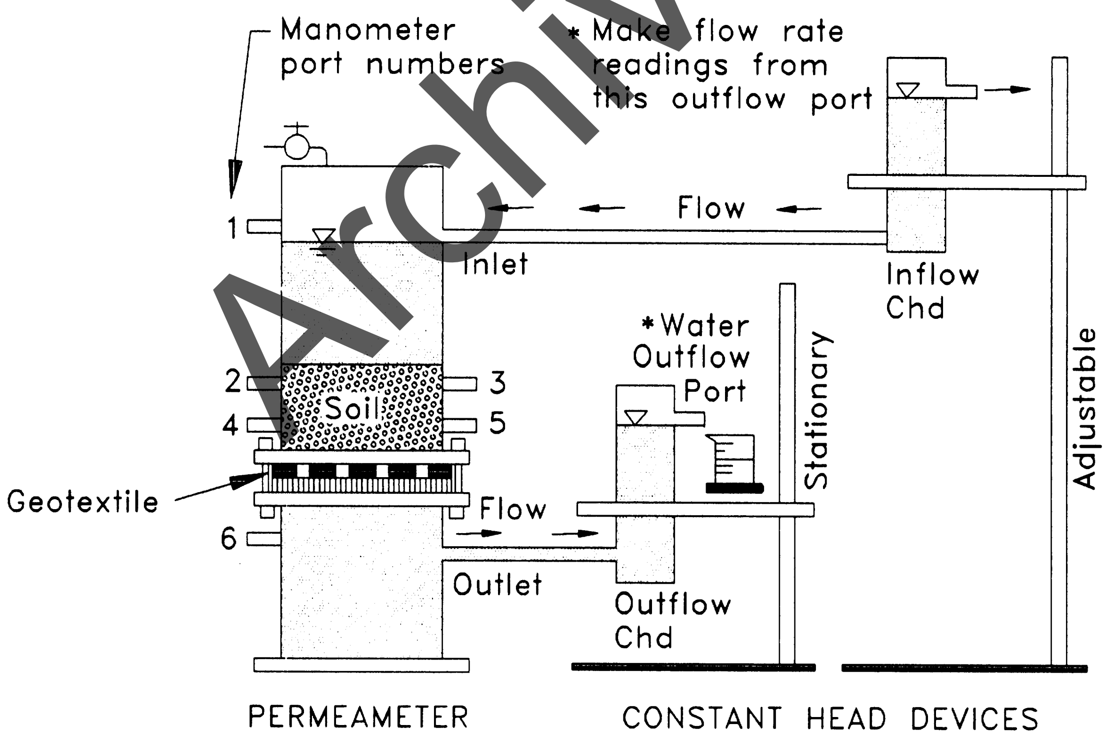


Figure 2-4 U.S. Army Corps of Engineers gradient ratio test device.

25 mm) to the head loss across 50 mm of soil is termed the *gradient ratio*. As fine soil particles adjacent to the geotextile become trapped inside or blind the surface, the gradient ratio will increase. A gradient ratio less than 3 is recommended by the U.S. Army Corps of Engineers (1977), based upon limited testing with severely gap-graded soils. Because the test is conducted in a rigid-wall permeameter, it is most appropriate for sandy and silty soils with $k \geq 10^{-7}$ m/s.

For soils with permeabilities less than about 10^{-7} m/s, long-term filtration tests should be conducted in a flexible wall or triaxial type apparatus to insure that flow is through the soil rather than along the sides of the specimen. The soil flexible wall test is ASTM D 5084, while the Hydraulic Conductivity Ratio (HCR) test (ASTM D 5567) has been suggested for geotextiles (see Section 1.5). Unfortunately, neither test is able to measure the permeability near the soil-geotextile interface nor determine changes in permeability and hydraulic gradient within the soil sample itself - a serious disadvantage (Fischer, 1994). Fortunately, very fine-grained, low-permeability soils rarely present a filtration problem unless they are dispersive (Sherard and Decker, 1977) or subject to hydraulic fracturing, such as might occur in dams under high hydraulic gradients (Sherard, 1986).

Again, we emphasize that these filtration tests are *performance tests*. They must be conducted on samples of project site soil by the specifying agency or its representative. These tests are the responsibility of the engineer because manufacturers generally do not have soil laboratories or samples of on-site soils. Therefore, realistically, the manufacturers are unable to certify the clogging resistance of a geotextile.

For *less critical/less severe* conditions, a simple way to avoid clogging, especially with silty soils, is to allow fine particles already in suspension to pass through the geotextile. Then the *bridge network* (Figure 2-2) formed by the larger particles retains the smaller particles. The bridge network should develop rather quickly, and the quantity of fine particles actually passing through the geotextile is relatively small. This is why the less critical/less severe clogging resistance criteria requires an AOS (O_{95}) sufficiently larger than the finer soil particles (D_{15}). Those are the particles that will pass through the geotextile. Unfortunately, the AOS value only indicates the size and not the number of O_{95}-sized holes available. Thus, the finer soil particles will be retained by the smaller holes in the geotextile, and if there are sufficient fines, a significant reduction in flow rate can occur.

Consequently, to control the number of holes in the geotextile, it may be desirable to increase other qualifiers such as the porosity and open area requirements. There should always be a sufficient number of holes in the geotextile to maintain permeability and drainage, even if some of them clog.

It should be pointed out that some soil types and gradations, may result in calculated AOS values that cannot reasonably be met by any available product. In these cases, the design must be modified accordingly to accommodate available products or possibly use multistage filters. In either case, performance tests should then be performed on the selected system.

2.3-4 Survivability and Endurance Criteria

To be sure the geotextile will survive the construction process, certain geotextile strength and endurance properties are required for filtration and drainage applications. These **minimum** requirements are given in Table 2-2. Note that stated values are for less critical/less severe applications.

It is important to realize that these **minimum** survivability values are not based on any systematic research, but on the properties of existing geotextiles which are known to have performed satisfactorily in drainage applications. The values are meant to serve as guidelines for inexperienced users in selecting geotextiles for routine projects. They are not intended to replace site-specific evaluation, testing, and design.

Geotextile endurance relates to its longevity. Geotextiles have been shown to be basically inert materials for most environments and applications. However, certain applications may expose the geotextile to chemical or biological activity that could drastically influence its filtration properties or durability. For example, in drains, granular filters and geotextiles can become chemically clogged by iron or carbonate precipitates, and biologically clogged by algae, mosses, etc. Biological clogging is a potential problem when filters and drains are periodically inundated then exposed to air. Excessive chemical and biological clogging can significantly influence filter and drain performance. These conditions are present, for example, in landfills.

Biological clogging potential can be examined with ASTM D 1987, Standard Test Method for Biological Clogging of Geotextile or Soil/Geotextile Filters (1991). If biological clogging is a concern, a higher-porosity geotextile may be used, and/or the drain design and operation can include an inspection and maintenance program to flush the drainage system.

TABLE 2-2
GEOTEXTILE STRENGTH PROPERTY REQUIREMENTS[1,2,3,4] FOR DRAINAGE GEOTEXTILES
(after AASHTO, 1997)

Property	ASTM Test Method	Units	Geotextile Class 2[5] Elongation < 50%[6]	Geotextile Class 2[5] Elongation ≥ 50%[6]
Grab Strength	D 4632	N	1100	700
Sewn Seam Strength[7]	D 4632	N	990	630
Tear Strength	D 4533	N	400[8]	250
Puncture Strength	D 4833	N	400	250
Burst Strength	D 3786	kPa	2700	1300

NOTES:
1. Acceptance of geotextile material shall be based on ASTM D 4759.
2. Acceptance shall be based upon testing of either conformance samples obtained using Procedure A of ASTM D 4354, or based on manufacturer's certifications and testing of quality assurance samples obtained using Procedure B of ASTM D 4354.
3. Minimum; use value in weaker principal direction. All numerical values represent minimum average roll value (*i.e.*, test results from any sampled roll in a lot shall meet or exceed the minimum values in the table). Lot samples according to ASTM D 4354.
4. Woven slit film geotextiles will not be allowed.
5. Default geotextile selection. The engineer may specify a Class 3 geotextile (see Appendix D) for trench drain applications based on one or more of the following:
   a) The engineer has found Class 3 geotextiles to have sufficient survivability based on field experience.
   b) The engineer has found Class 3 geotextiles to have sufficient survivability based on laboratory testing and visual inspection of a geotextile sample removed from a field test section constructed under anticipated field conditions.
   c) Subsurface drain depth is less than 2 m, drain aggregate diameter is less than 30 mm and compaction requirement is equal to or less than 95% of AASHTO T-99.
6. As measured in accordance with ASTM D 4632.
7. When seams are required. Values apply to both field and manufactured seams.
8. The required MARV tear strength for woven monofilament geotextiles is 250 N.

2.4 DRAINAGE SYSTEM DESIGN GUIDELINES

In this section, step-by-step design procedures are given. As with a chain, the integrity of the resulting design will depend on its weakest link; thus, no steps should be compromised or omitted.

STEP 1. Evaluate the critical nature and site conditions (see Table 2.1) of the application.

Reasonable judgment should be used in categorizing a project, since there may be a significant cost difference for geotextiles required for critical/severe conditions. Final selection should *not* be based on the lowest material cost alone, nor should costs be reduced by eliminating laboratory soil-geotextile performance testing, if such testing is appropriate.

STEP 2. Obtain soil samples from the site and:

A. Perform grain size analyses.

- Calculate $C_u = D_{60}/D_{10}$ (Eq. 2 - 3)
- Select the *worst case* soil for retention (*i.e.*, usually the soil with smallest B *x* D_{85})

NOTE: When the soil contains particles 25 mm and larger, use only the gradation of soil passing the 4.75 mm sieve in selecting the geotextile (*i.e.*, scalp off the +4.75 mm material).

B. Perform field or laboratory permeability tests.

- Select worst case soil (*i.e.*, soil with highest coefficient of permeability, k).
- The permeability of clean sands with 0.1 mm < D_{10} < 3 mm and C_u < 5 can be estimated by the Hazen formula, $k = (D_{10})^2$ (k in cm/s; D_{10} in mm). This formula should **not** be used for soils with appreciable fines.

C. Select drainage aggregate.

- Use free-draining, open-graded material and determine its permeability (*e.g.*, Figure 2-5). If possible, sharp, angular aggregate should be avoided. If it must be used, then a geotextile meeting the property requirements for high survivability in Table 2-2 should be specified. For an accurate design cost comparison, compare cost of open-graded aggregate with select well-graded, free-draining filter aggregate.

STEP 3. Calculate anticipated flow into and through drainage system and dimension the system. Use collector pipe to reduce size of drain.

A. General Case

Use Darcy's Law

$$q = k\,i\,A \qquad [2 - 13]$$

where:

q	=	infiltration rate (L^3/T)
k	=	effective permeability of soil (from Step 2B above) (L/T)
i	=	average hydraulic gradient in soil and in drain (L/L)
A	=	area of soil and drain material normal to the direction of flow (L^2)

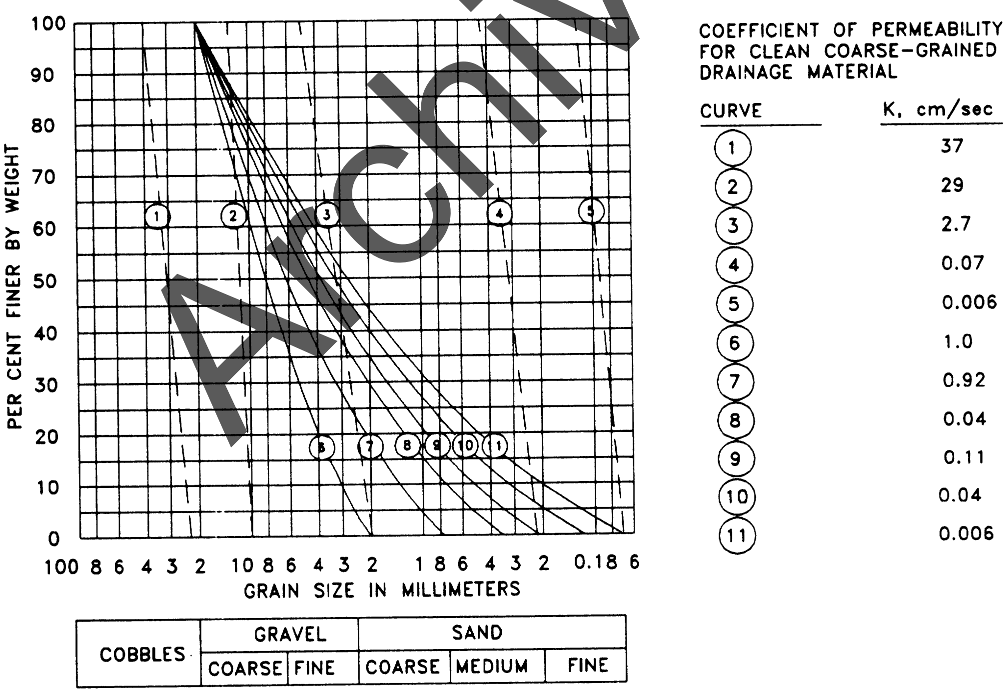


Figure 2-5 Typical gradations and Darcy permeabilities of several aggregate and graded filter materials (U.S. Navy, 1982).

Use conventional flow net analysis to calculate the hydraulic gradient (Cedergren, 1977) and Darcy's Law for estimating infiltration rates into drain; then use Darcy's Law to design drain (*i.e.*, calculate cross-sectional area A for flow through open-graded aggregate). Note that typical values of hydraulic gradients in the soil adjacent to a geotextile filter (Giroud, 1988) are:

- $i < 1$ for drainage under roads, embankments, slopes, etc., when the main source of water is precipitation; and
- $i = 1.5$ in the case of drainage trenches and vertical drains behind walls.

B. Specific Drainage Systems

Estimates of surface infiltration, runoff infiltration rates, and drainage dimensions can be determined using accepted principles of hydraulic engineering (Moulton, 1980). Specific references are:

1. *Flow into trenches* -- Mansur and Kaufman (1962)
2. *Horizontal blanket drains* -- Cedergren (1977)
3. *Slope drains* -- Cedergren (1977)

STEP 4. Determine geotextile requirements.

A. Retention Criteria

From Step 2A, obtain D_{85} and C_u; then determine largest pore size allowed.

$$AOS \leq B\,D_{85} \qquad \text{(Eq. 2 - 1)}$$

where:

B = 1 for a conservative design. For a less-conservative design, and for ≤ 50% passing 0.075 mm sieve:

$$B = 1 \quad \text{for } C_u \leq 2 \text{ or } \geq 8 \qquad \text{(Eq. 2 - 2a)}$$
$$B = 0.5\,C_u \quad \text{for } 2 \leq C_u \leq 4 \qquad \text{(Eq. 2 - 2b)}$$
$$B = 8/C_u \quad \text{for } 4 < C_u < 8 \qquad \text{(Eq. 2 - 2c)}$$

and, for ≥ 50% passing 0.075 mm sieve:

B = 1 for wovens,
B = 1.8 for nonwovens,
and AOS (geotextile) ≤ 0.3 mm (Eq. 2 - 5)

NOTE: Soils with a C_u of greater than 20 may be unstable (see section 2.3-1.c): if so, performance tests should be conducted to select suitable geotextiles.

B. Permeability/Permittivity Criteria

1. Less Critical/Less Severe

$k_{geotextile} \geq k_{soil}$ (Eq. 2 - 7a)

2. Critical/Severe

$k_{geotextile} \geq 10\ k_{soil}$ (Eq. 2 - 7b)

3. Permittivity Requirements

$\psi \geq 0.5\ sec^{-1}$ for < 15% passing 0.075 mm (Eq. 2 - 8a)
$\psi \geq 0.2\ sec^{-1}$ for 15 to 50% passing 0.075 mm (Eq. 2 - 8b)
$\psi \geq 0.1\ sec^{-1}$ for > 50% passing 0.075 mm (Eq. 2 - 8c)

4. Flow Capacity Requirement

$q_{required} = q_{geotextile}/(A_g/A_t)$, or (Eq. 2 - 9)

$$(k_{geotextile}/t)\ h\ A_g \geq q_{required} \qquad [2 - 14]$$

where:

$q_{required}$ is obtained from STEP 3B (Eq. 2-14) above;
$k_{geotextile}/t$ = ψ = permittivity;
t = geotextile thickness;
h = average head in field;
A_g = geotextile area available for flow (*i.e.*, if 80% of geotextile is covered by the wall of a pipe, A_g = 0.2 x total area); and
A_t = total area of geotextile.

C. Clogging Criteria

1. Less Critical/Less Severe
   a. From Step 2A obtain D_{15}; then determine minimum pore size requirement from
   $O_{95} \geq 3\ D_{15}$, for $C_u > 3$ (Eq. 2 - 10)
   b. Other qualifiers:
   Nonwovens:
   Porosity (geotextile) ≥ 50% (Eq. 2 - 11)

Wovens:

Percent open area $\geq$ 4% (Eq. 2 - 12)

Alternative: Run filtration tests

2. Critical/Severe

Select geotextiles that meet retention, permeability, and survivability criteria, as well as the criteria in Step 4C.1 above, and perform a filtration test.

Suggested filtration test for sandy and silty soils is the gradient ratio test. The hydraulic conductivity ratio test is recommended by some people for fine-grained soils, but as noted in Section 2.3-3, the test has serious disadvantages.

Alternative: Long-term filtration tests, F^3 tests, etc.

NOTE: Experience is required to obtain reproducible results from the gradient ratio test. See Fischer (1994) and Maré (1994).

D. Survivability

Select geotextile properties required for survivability from Table 2-2. Add durability requirements if appropriate.

STEP 5. Estimate costs.

Calculate the pipe size (if required), the volume of aggregate, and the area of the geotextile. Apply appropriate unit cost values.

Pipe (if required) (/m)	________________
Aggregate (/m^3)	________________
Geotextile (/m^2)	________________
Geotextile placement (/m^2)	________________
Construction (LS)	________________
Total Cost:	________________

STEP 6. Prepare specifications.

Include for the geotextile:

A. General requirements

B. Specific geotextile properties
C. Seams and overlaps
D. Placement procedures
E. Repairs
F. Testing and placement observation requirements

See Sections 1.6 and 2.7 for specification details.

STEP 7. Collect samples of aggregate and geotextile before acceptance.

STEP 8. Monitor installation during and after construction.

STEP 9. Observe drainage system during and after storm events.

2.5 DESIGN EXAMPLE

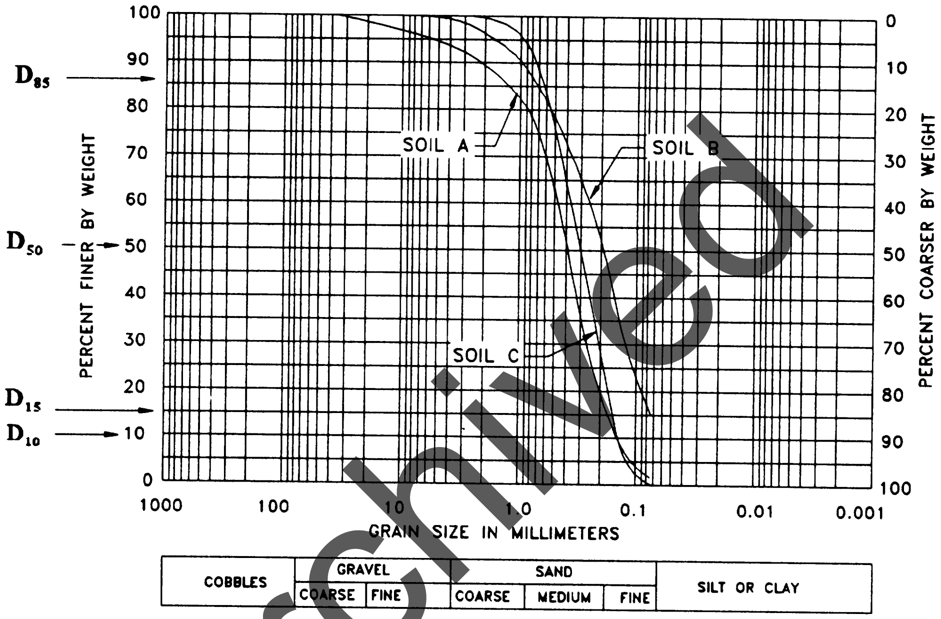
<u>DEFINITION OF DESIGN EXAMPLE</u>

- Project Description: drains to intercept groundwater are to be placed adjacent to a two-lane highway
- Type of Structure: trench drain
- Type of Application: geotextile wrapping of aggregate drain stone
- Alternatives: i) graded soil filter between aggregate and soil being drained; or ii) geotextile wrapping of aggregate

<u>GIVEN DATA</u>

- site has a high groundwater table
- drain is to prevent seepage and shallow slope failures, which are currently a maintenance problem
- depth of trench drain is 1 meter
- soil samples along the proposed drain alignment are nonplastic
- gradations of three *representative* soil samples along the proposed drain alignment

SIEVE SIZE (mm)	PERCENT PASSING, BY WEIGHT		
	Sample A	Sample B	Sample C
25	99	100	100
13	97	100	100
4.76	95	100	100
1.68	90	96	100
0.84	78	86	93
0.42	55	74	70
0.15	10	40	11
0.074	1	15	0



Grain Size Distribution Curve

DEFINE

A. Geotextile function(s)

B. Geotextile properties required

C. Geotextile specification

SOLUTION

A. Geotextile function(s):

Primary	-	filtration
Secondary	-	separation

B. Geotextile properties required:
- apparent opening size (AOS)
- permittivity
- survivability

DESIGN

STEP 1. EVALUATE CRITICAL NATURE AND SITE CONDITIONS

From given data, assume that this is a noncritical application.
Soils are well-graded, hydraulic gradient is low for this type of application, and flow conditions are steady state for this type of application.

STEP 2. OBTAIN SOIL SAMPLES

A. GRAIN SIZE ANALYSES
Plot gradations of representative soils. The D_{60}, D_{10}, and D_{85} sizes from the gradation plot are noted in the table below for Samples A, B, and C. Determine uniformity coefficient, C_u, coefficient B, and the maximum AOS.

Worst case soil for retention (*i.e.*, smallest B *x* D_{85}) is **Soil C**, from the following table.

Soil Sample	$D_{60} \div D_{10} = C_u$	B =	AOS (mm) $\leq$ B *x* D_{85}
A	$0.48 \div 0.15 = 3.2$	$0.5\ C_u = 0.5 \times 3.2 = 1.6$	$1.6 \times 1.0 = 1.6$
B	$0.25 \div 0.06 = 4.2$	$8 \div C_u = 8 \div 4.2 = 1.9$	$1.9 \times 0.75 = 1.4$
C	$0.36 \div 0.14 = 2.6$	$0.5\ C_u = 0.5 \times 2.6 = 1.3$	$1.3 \times 0.55 = 0.72$

B. PERMEABILITY TESTS
Noncritical application, drain will be conservatively designed with an estimated permeability.

The largest D_{10} controls permeability; therefore, Soil A with $D_{10} = 0.15$ mm controls. Therefore,

$$k \approx (D_{10})^2 = (0.15)^2 = 2\,(10)^{-2}\ \text{cm/s} = 2\,(10)^{-4}\ \text{m/s}$$

C. SELECT DRAIN AGGREGATE
Assume drain stone is a rounded aggregate.

STEP 3. DIMENSION DRAIN SYSTEM

Determine depth and width of drain trench and whether a pipe is required to carry flow - details of which are not included within this example.

STEP 4. DETERMINE GEOTEXTILE REQUIREMENTS

A. RETENTION CRITERIA
Sample C controls (see table above), therefore, **AOS ≤ 0.72 mm**

B. PERMEABILITY CRITERIA
From given data, it has been judged that this application is a less critical/less severe application. Therefore, $k_{geotextile} \geq k_{soil}$

Soil C controls, therefore **$k_{geotextile} \geq 2\ (10)^{-4}$ m/sec**

Flow capacity requirements of the system - details of which are not included within this example.

C. PERMITTIVITY CRITERIA
All three soils have < 15% passing the 0.075 mm, therefore **$\psi \geq 0.5$ sec^{-1}**

D. CLOGGING CRITERIA
From given data, it has been judged that this application is a less critical/less severe application, and Soils A and B have a C_u greater than 3. Therefore, for soils A and B, $O_{95} \geq 3\ D_{15}$

$O_{95} \geq$ 3 *x* 0.15 = 0.45 mm for Sample A
3 *x* 0.075 = 0.22 mm for Sample B

Soil A controls [Note that sand size particles typically don't create clogging problems, therefore, Soil B could have been used as the design control.], therefore, **AOS ≥ 0.45 mm**

For Soil C, a geotextile with the maximum AOS value determined from the retention criteria should be used. Therefore **AOS ≈ 0.72 mm**

Also,
nonwoven porosity ≥ 50%
and
woven percent open area ≥ 4%

For the primary function of filtration, the geotextile should have 0.45 mm ≤ AOS ≤ 0.72 mm; and $k_{geotextile} \geq 2\ (10)^{-2}$ cm/sec and, $\psi \geq 0.5$ sec^{-1}. Woven slit film geotextiles are not allowed.

E. SURVIVABILITY
From Table 2-2, the following **minimum** values are recommended:

For Survivability, the geotextile shall have the following minimum values (values are MARV) -

	Woven Geotextile	Nonwoven Geotextile
Grab Strength	1100 N	700 N
Sewn Seam Strength	990 N	630 N
Tear Strength	400* N	250 N
Puncture Strength	400 N	250 N
Trapezoidal Tear	2700 N	1300 N

*250 N for monofilament geotextiles

NOTE: With lightweight compaction equipment and field inspection, Class 3 geotextile (see Appendix D) could be used.

Complete Steps 5 through 9 to finish design.

STEP 5. ESTIMATE COSTS

STEP 6. PREPARE SPECIFICATIONS

STEP 7. COLLECT SAMPLES

STEP 8. MONITOR INSTALLATION

STEP 9. OBSERVE DRAIN SYSTEM DURING AND AFTER STORM EVENTS

2.6 COST CONSIDERATIONS

Determining the cost effectiveness of geotextiles versus conventional drainage systems is a straightforward process. Simply compare the cost of the geotextile with the cost of a conventional granular filter layer, while keeping in mind the following:

- *Overall material costs including a geotextile versus a conventional system* - For example, the geotextile system will allow the use of poorly graded (less-select) aggregates, which may reduce the need for a collector pipe, provided the amount of fines is small (Q decreases considerably if the percent passing the 0.075 mm sieve is greater than 5%, even in gravel).
- *Construction requirements* - There is, of course, a cost for placing the geotextile; but in most cases, it is less than the cost of constructing dual-layered, granular filters, for example, which are often necessary with conventional filters and fine-grained soils.
- *Possible dimensional design improvements* - If an open-graded aggregate is used (especially with a collector pipe), a considerable reduction in the physical dimensions of the drain can be made without a decrease in flow capacity. This size reduction also reduces the volume of the excavation, the volume of filter material required, and the construction time necessary per unit length of drain.

In general, the cost of the geotextile material in drainage applications will typically range from \$1.00 to \$1.50 per square meter, depending upon the type specified and quantity ordered. Installation costs will depend upon the project difficulty and contractor's experience; typically, they range from \$0.50 to \$1.50 per square meter of geotextile. Higher costs should be anticipated for below-water placement. Labor installation costs for the geotextile are easily repaid because construction can proceed at a faster pace, less care is needed to prevent segregation and contamination of granular filter materials, and multilayered granular filters are typically not necessary.

2.7 SPECIFICATIONS

The following guide specification is provided as an example. It is a combination of the AASHTO M288 (1997) geotextile material specification and its accompanying construction/installation guidelines; developed for routine drainage and filtration applications. The actual hydraulic and physical properties of the geotextile must be selected by considering of the nature of the project (critical/less critical), hydraulic conditions (severe/less severe), soil conditions at the site, and construction and installation procedures appropriate for the project.

SUBSURFACE DRAINAGE GEOTEXTILES
(after AASHTO M288, 1997)

1. SCOPE

1.1 *Description.* This specification is applicable to placing a geotextile against the soil to allow long-term passage of water into a subsurface drain system retaining the in situ soils. The primary function of the geotextile in subsurface drainage applications is filtration. Geotextile filtration properties are a function of the in situ soil gradation, plasticity, and hydraulic conditions.

2. REFERENCED DOCUMENTS

2.1 *AASHTO Standards*

T88	Particle Size Analysis of Soils
T90	Determining the Plastic Limit and Plasticity Index of Soils
T99	The Moisture-Density Relationships of Soils Using a 2.5 kg Rammer and a 305 mm Drop

2.2 *ASTM Standards*

D 123	Standard Terminology Relating to Textiles
D 276	Test Methods for Identification of Fibers in Textiles
D 3786	Test Method for Hydraulic Burst Strength of Knitted Goods and Nonwoven Fabrics, Diaphragm Bursting Strength Tester Method
D 4354	Practice for Sampling of Geosynthetics for Testing
D 4355	Test Method for Deterioration of Geotextiles from Exposure to Ultraviolet Light and Water (Xenon Arc Type Apparatus)
D 4439	Terminology for Geosynthetics
D 4491	Test Methods for Water Permeability of Geotextiles by Permittivity
D 4632	Test Method for Grab Breaking Load and Elongation of Geotextiles
D 4751	Test Method for Determining Apparent Opening Size of a Geotextile
D 4759	Practice for Determining the Specification Conformance of Geosynthetics
D 4833	Test Method for Index Puncture Resistance of Geotextiles, Geomembranes and Related Products
D 4873	Guide for Identification, Storage, and Handling of Geotextiles
D 5141	Test Method to Determine Filtering Efficiency and Flow Rate for Silt Fence Applications Using Site Specific Soil

3. PHYSICAL AND CHEMICAL REQUIREMENTS

3.1 Fibers used in the manufacture of geotextiles and the threads used in joining geotextiles by sewing, shall consist of long chain synthetic polymers, composed of at least 95% by weight polyolefins or polyesters. They shall be formed into a stable network such that the filaments or yarns retain their dimensional stability relative to each other, including selvages.

3.2 *Geotextile Requirements.* The geotextile shall meet the requirements of following Table. Woven slit film geotextiles (*i.e.*, geotextiles made from yarns of a flat, tape-like character) will not be allowed. All numeric values in the following table, except AOS, represent minimum average roll values (MARV) in the weakest principal direction (*i.e.*, average test results of any roll in a lot sampled for conformance or quality assurance testing shall meet or exceed the minimum values). Values for AOS represent maximum average roll values.

NOTE: The property values in the following table represent default values which provide for sufficient geotextile survivability under most conditions. Minimum property requirements may be reduced when sufficient survivability information is available [see Note 2 of Table 2-2 and Appendix D]. The Engineer may also specify properties different from those listed in the following Table based on engineering design and experience.

Subsurface Drainage Geotextile Requirements

Property	ASTM Test Method	Units	Elongation[1]		
			< 50%[1]	≥ 50%[1]	
Grab Strength	D 4632	N	1100	700	
Sewn Seam Strength[2]	D 4632	N	990	630	
Tear Strength	D 4533	N	400[3]	250	
Puncture Strength	D 4833	N	400	250	
Burst Strength	D 3786	kPa	2700	1300	
			Percent In Situ Passing 0.075 mm Sieve[4]		
			< 15	15 to 50	> 50
Permittivity	D 4491	sec^{-1}	0.5	0.2	0.1
Apparent Opening Size	D 4751	mm	0.43	0.25	0.22[5]
Ultraviolet Stability	D 4355	%	50% after 500 hours of exposure		

NOTES:

(1) As measured in accordance with ASTM D 4632.

(2) When sewn seams are required.

(3) The required MARV tear strength for woven monofilament geotextiles is 250 N.

(4) Based on grain size analysis of in situ soil in accordance with AASHTO T88.

(5) For cohesive soils with a plasticity index greater than 7, geotextile maximum average roll value for apparent opening size is 0.30 mm.

4. CERTIFICATION

4.1 The Contractor shall provide to the Engineer a certificate stating the name of the manufacturer, product name, style number, chemical composition of the filaments or yarns and other pertinent information to fully describe the geotextile.

4.2 The Manufacturer is responsible for establishing and maintaining a quality control program to assure compliance with the requirements of the specification. Documentation describing the quality control program shall be made available upon request.

4.3 The Manufacturer's certificate shall state that the furnished geotextile meets MARV requirements of the specification as evaluated under the Manufacturer's quality control program. The certificate shall be attested to be a person having legal authority to bind the Manufacturer.

4.4 Either mislabeling or misrepresentation of materials shall be reason to reject those geotextile products.

5. SAMPLING, TESTING, AND ACCEPTANCE

5.1 Geotextiles shall be subject to sampling and testing to verify conformance with this specification. Sampling for testing shall be in accordance with ASTM D 4354. Acceptance shall be based on testing of either conformance samples obtained using Procedure A of ASTM D 4354, or based on manufacturer's certifications and testing of quality assurance samples obtained using Procedure B of ASTM D 4354. A lot size for conformance or quality assurance sampling shall be considered to be the shipment quantity of the given product or a truckload of the given product, whichever is smaller.

5.2 Testing shall be performed in accordance with the methods referenced in this specification for the indicated application. The number of specimens to test per sample is specified by each test method. Geotextile product acceptance shall be based on ASTM D 4759. Product acceptance is determined by comparing the average test results of all specimens within a given sample to the specification MARV. Refer to ASTM D 4759 for more details regarding geotextile acceptance procedures.

6. SHIPMENT AND STORAGE

6.1 Geotextile labeling, shipment, and storage shall follow ASTM D 4873. Product labels shall clearly show the manufacturer or supplier name, style number, and roll number. Each shipping document shall include a notation certifying that the material is in accordance with the manufacturer's certificate.

6.2 Each geotextile roll shall be wrapped with a material that will protect the geotextile from damage due to shipment, water, sunlight, and contaminants. The protective wrapping shall be maintained during periods of shipment and storage.

6.3 During storage, geotextile rolls shall be elevated off the ground and adequately covered to protect them from the following: site construction damage, precipitation, extended ultraviolet radiation including sunlight, chemicals that are strong acids or strong bases, flames including welding sparks, temperatures in excess of 71°C (160°F), and any other environmental condition that may damage the physical property values of the geotextile.

7. CONSTRUCTION

7.1 *General.* Atmospheric exposure of geotextiles to the elements following lay down shall be a maximum of 14 days to minimize damage potential.

7.2 *Seaming.*

a. If a sewn seam is to be used for the seaming of the geotextile, the thread used shall consist of high strength polypropylene, or polyester. Nylon thread shall not be used. For erosion control applications, the thread shall also be resistant to ultraviolet radiation. The thread shall be of contrasting color to that of the geotextile itself.

b. For seams which are sewn in the field, the Contractor shall provide at least a 2 m length of sewn seam for sampling by the Engineer before the geotextile is installed. For seams which are sewn in the factory, the Engineer shall obtain samples of the factory seams at random from any roll of geotextile which is to be used on the project.

b.1 For seams that are field sewn, the seams sewn for sampling shall be sewn using the same equipment and procedures as will be used for the production of seams. If seams are to be sewn in both the machine and cross machine directions, samples of seams from both directions shall be provided.

b.2 The seam assembly description shall be submitted by the Contractor along with the sample of the seam. The description shall include the seam type, stitch type, sewing thread, and stitch density.

7.3 *Trench.* Trench excavation shall be done in accordance with details of the project plans. In all instances excavation shall be done in such a way so as to prevent large voids from occurring in the sides and bottom of the trench. The graded surface shall be smooth and free and debris.

7.4 *Geotextile Placement.*

a. In placement of the geotextile for drainage applications, the geotextile shall be placed loosely with no wrinkles or folds, and with not void spaces between the geotextile and the ground surface. Successive sheets of geotextiles shall be overlapped a minimum of 300 mm, with the upstream sheet overlapping the downstream sheet.

a.1 In trenches equal to or greater than 300 mm in width, after placing the drainage aggregate the geotextile shall be folded over the top of the backfill material in a manner to produce a minimum overlap of 300 mm. In trenches less than 300 mm but greater than 100 mm wide, the overlap shall be equal to the width of the trench. Where the trench is less than 100 mm the geotextile overlap shall be sewn or otherwise bonded. All seams shall be subject to the approval of the Engineer.

a.2 Should the geotextile be damaged during installation, or drainage aggregate placement, a geotextile patch shall be placed over the damaged area extending beyond the damaged area a distance of 300 mm, or the specified seam overlap, whichever is greater.

7.5 *Drainage Aggregate*

a. Placement of drainage aggregate should proceed immediately following placement of the geotextile. The geotextile should be covered with a minimum of 300 mm of loosely placed aggregate prior to compaction. If a perforated collector pipe is to be installed in the trench, a bedding layer of drainage aggregate should be placed below the pipe, with the remainder of the aggregate placed to the minimum required construction depth.

a.1 The aggregate should be compacted with vibratory equipment to a minimum of 95% Standard AASHTO density unless the trench is required for structural support. If higher compactive effort is required, a Class 1 geotextile as per Table 1 of the M288 Specification is needed.

8. METHOD OF MEASUREMENT

8.1 The geotextile shall be measured by the number of square meters computed from the payment lines shown on the plans or from payment lines established in writing by the Engineer. This excludes seam overlaps, but shall include geotextiles used in crest and toe of slope treatments.

8.2 Slope preparation, excavation and backfill, bedding, and cover material are separate pay items.

9. BASIS OF PAYMENT

9.1 The accepted quantities of geotextile shall be paid for per square meter in place.

9.2 Payment will be made under:

Pay Item	Pay Unit
Subsurface Drainage Geotextile	Square Meter

2.8 INSTALLATION PROCEDURES

For all drainage applications, the following construction steps should be followed:

1. The surface on which the geotextile is to be placed should be excavated to design grade to provide a smooth, graded surface free of debris and large cavities.
2. Between preparation of the subgrade and construction of the system itself, the geotextile should be well-protected to prevent any degradation due to exposure to the elements.
3. After excavating to design grade, the geotextile should be cut (if required) to the desired width (including allowances for *non-tight* placement in trenches and overlaps of the ends of adjacent rolls) or cut at the top of the trench after placement of the drainage aggregate.
4. Care should be taken during construction to avoid contamination of the geotextile. If it becomes contaminated, it must be removed and replaced with new material.
5. In drainage systems, the geotextile should be placed with the machine direction following the direction of water flow; for pavements, the geotextile should be parallel to the roadway. It should be placed loosely (not taut), but with no wrinkles or folds. Care should be taken to place the geotextile in intimate contact with the soil so that no void spaces occur behind it.

6. The ends for subsequent rolls and parallel rolls of geotextile should be overlapped a minimum of 0.3 in roadways and 0.3 to 0.6 m in drains, depending on the anticipated severity of hydraulic flow and the placement conditions. For high hydraulic flow conditions and heavy construction, such as with deep trenches or large stone, the overlaps should be increased. For large open sites using base drains, overlaps should be pinned or anchored to hold the geotextile in place until placement of the aggregate. Upstream geotextile should always overlap over downstream geotextile.
7. To limit exposure of the geotextile to sunlight, dirt, damage, etc., placement of drainage or roadway base aggregate should proceed immediately following placement of the geotextile. The geotextile should be covered with a minimum of 0.3 m of loosely placed aggregate prior to compaction. If thinner lifts are used, higher survivability fabrics may be required. For drainage trenches, at least 0.1 m of drainage stone should be placed as a bedding layer below the slotted collector pipe (if required), with additional aggregate placed to the minimum required construction depth. Compaction is necessary to seat the drainage system against the natural soil and to reduce settlement within the drain. The aggregate should be compacted with vibratory equipment to a minimum of 95% Standard AASHTO T99 density unless the trench is required for structural support. If higher compactive efforts are required, the geotextiles meeting the property values listed under the high survivability category in Table 2-2 should be utilized.
8. After compaction, for trench drains, the two protruding edges of the geotextile should be overlapped at the top of the compacted granular drainage material. A minimum overlap of 0.3 m is recommended to ensure complete coverage of the trench width. The overlap is important because it protects the drainage aggregate from surface contamination. After completing the overlap, backfill should be placed and compacted to the desired final grade.

A schematic of the construction procedures for a geotextile-lined underdrain trench is shown in Figure 2-6. Construction photographs of an underdrain trench are shown in Figure 2-7, and diagrams of geosynthetic placement beneath a permeable roadway base are shown in Figure 2-8.

2.9 FIELD INSPECTION

The field inspector should review the field inspection guidelines in Section 1.7. Special attention should be given to aggregate placement and potential for geotextile damage. Also, maintaining the appropriate geotextile overlap at the top of the trench and at roll ends is especially important.

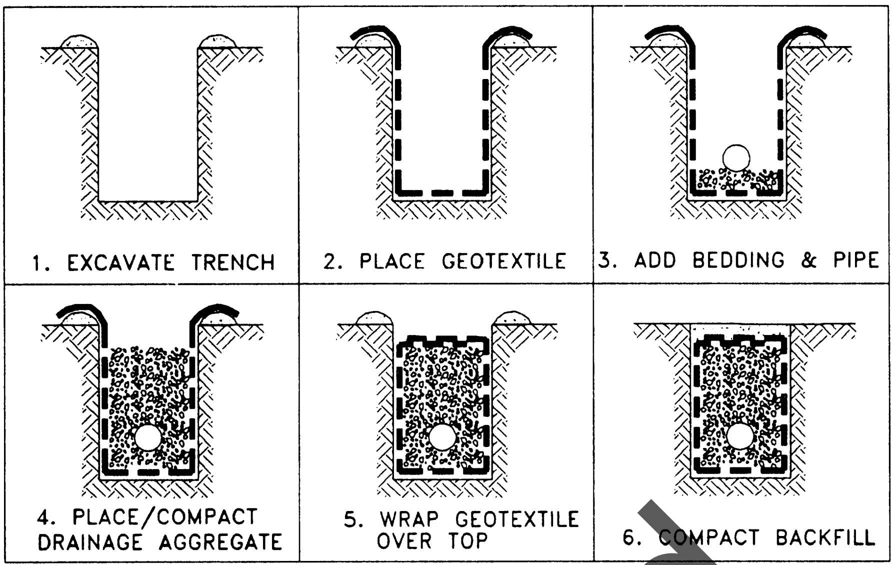


Figure 2-6 Construction procedure for geotextile-lined underdrains.

2.10 ADDITIONAL SELECTION CONSIDERATIONS

The late Dr. Allan Haliburton, a geotextile pioneer, noted that all geotextiles will work in some applications, but no one geotextile will work in all applications. Even though several types of geotextiles (monofilament wovens and an array of light- to heavy-weight nonwovens) may meet all of the desired design criteria, it may be preferable to use one type over another to enhance system performance. Selection will depend on the actual soil and hydraulic conditions, as well as the intended function of the design. Intuitively, the following considerations seem appropriate for the soil conditions given.

1. Graded gravels and coarse sands -- Very open monofilament or even multifilament wovens may be required to permit high rates of flow and low-risk of blinding.
2. Sands and gravels with less than 20% fines -- Open monofilament wovens and needlepunched nonwovens with large openings are preferable to reduce the risk of blinding. For thin, heat-bonded geotextiles and thick, needlepunched nonwoven geotextiles, filtration tests should be performed.
3. Soils with 20% to 60% fines -- Filtration tests should be performed on all types of geotextiles.

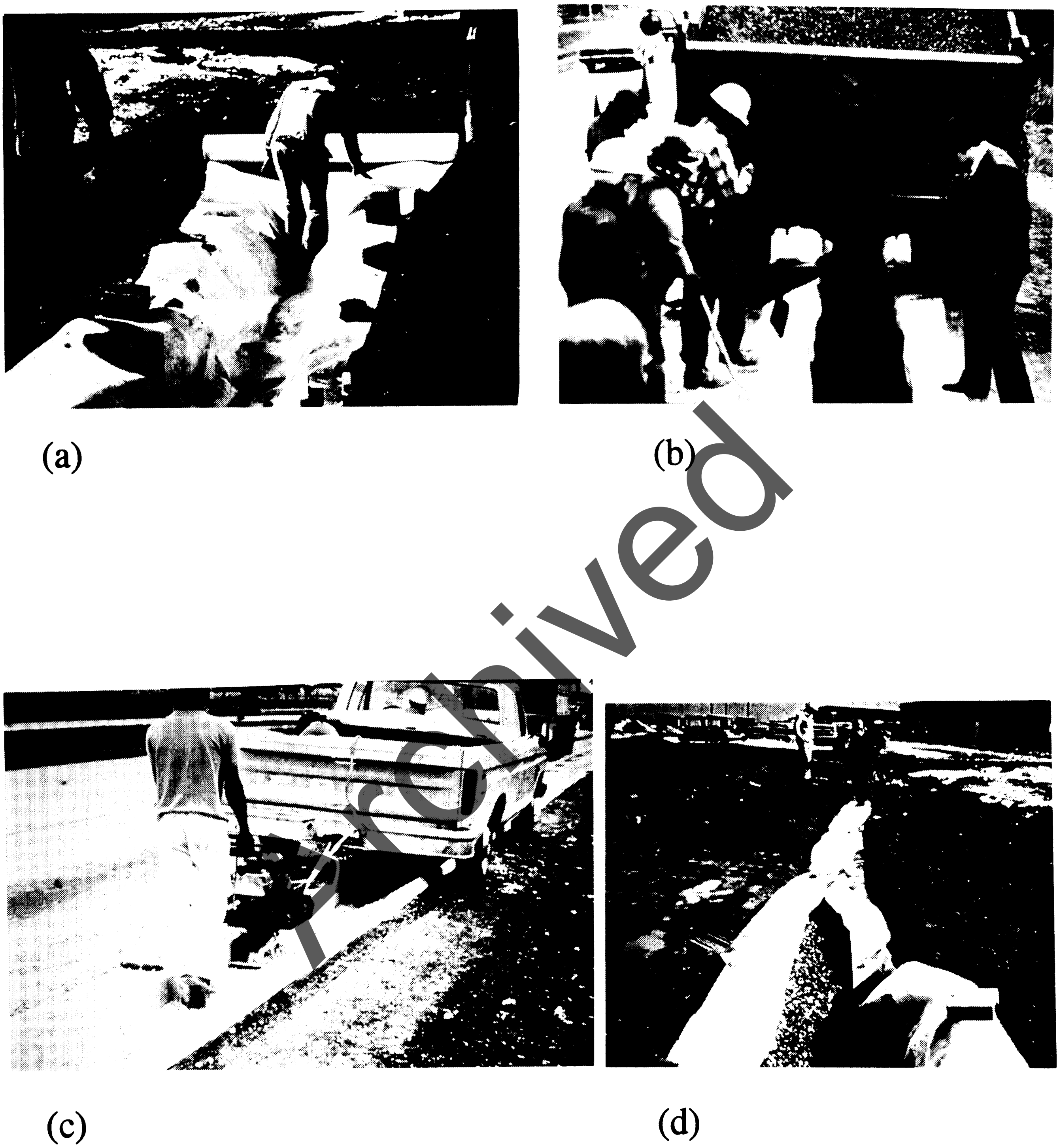


Figure 2-7 Construction of geotextile drainage systems: a.) geotextile placement in drainage ditch; b.) aggregate placement; c.) compaction of aggregate; and d.) geotextile overlap prior to final cover.

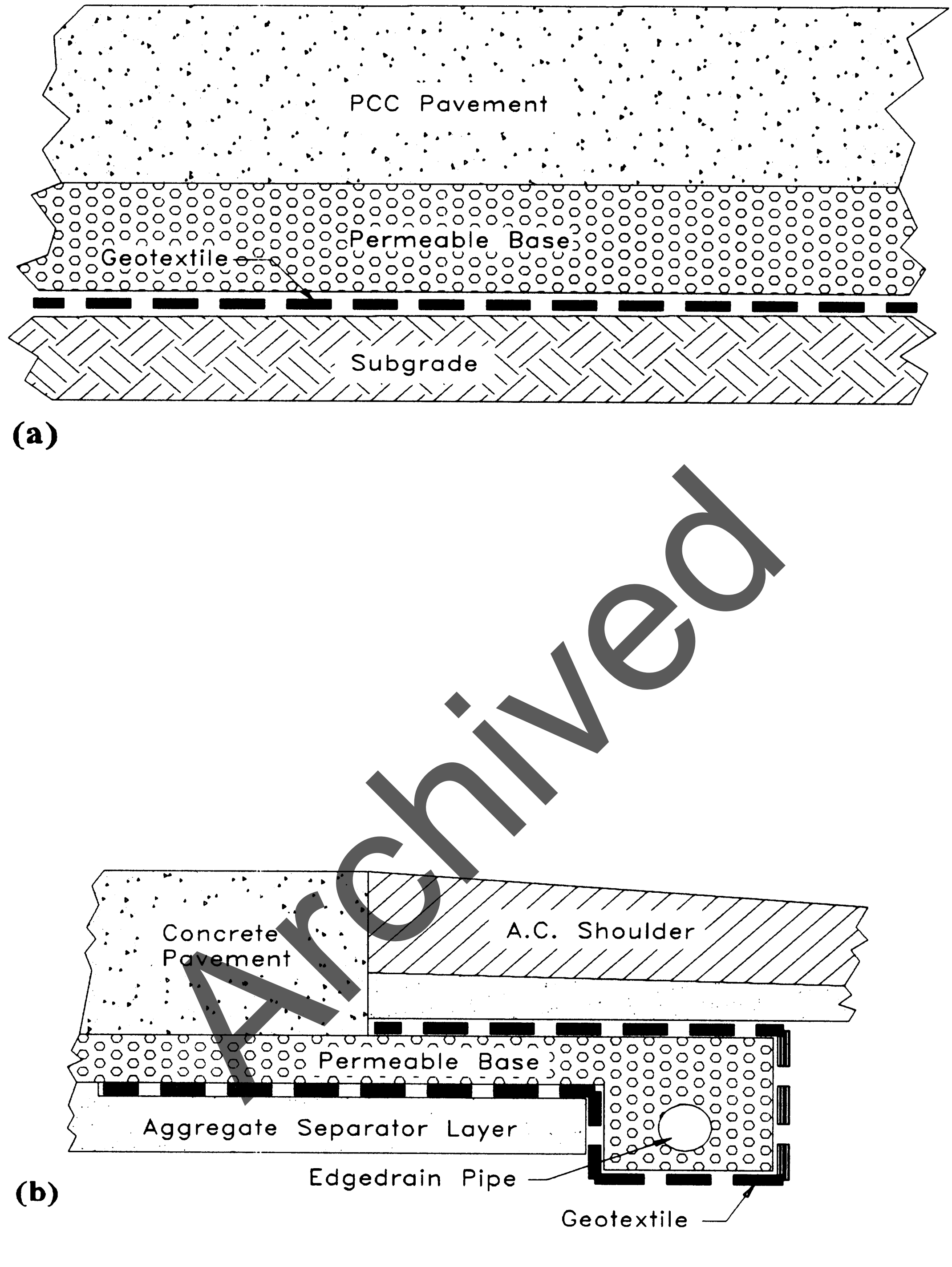


Figure 2-8 Construction geotextile filters and separators beneath permeable pavement base: a.) geotextile used as a separator; and b.) permeable base and edge drain combination. (Baumgardner, 1994)

4. Soils with greater than 60% fines -- Heavy-weight, needlepunched geotextiles and heat-bonded geotextiles tend to work best as fines will not pass. If blinding does occur, the permeability of the blinding cake would equal that of the soil.
5. Gap-graded cohesionless soils -- Consider using a uniform sand filter with a very open geotextile designed to allow fines to pass.
6. Silts with sand seams -- Consider using a uniform sand filter over the soil with a very open geotextile, designed to allow the silt to pass but to prevent movement of the filter sand; alternatively, consider using a heavy-weight (thick) needlepunched nonwoven directly against soil so water can flow laterally through the geotextile should it become locally clogged.

These general observations are not meant to serve as recommendations, but are offered to provide insight for selecting optimum materials. They are **not** intended to exclude other possible geotextiles that you may want to consider.

2.11 IN-PLANE DRAINAGE; PREFABRICATED GEOCOMPOSITE DRAINS

Geotextiles with high in-plane drainage ability and prefabricated geocomposite drains are potentially quite effective in several applications.

The ability of geotextiles to transmit water in the plane of the geotextile itself may be an added benefit in certain drainage applications where lateral transmission of water is desirable or where reduction of pore water pressures in the soil can be accelerated. These applications include interceptor drains, transmission of seepage water below pavement base course layers, horizontal and vertical strip drains to accelerate consolidation of soft foundation soils, dissipation of seepage forces in earth and rock slopes, as part of chimney drains in earth dams, dissipaters of pore water pressures in embankments and fills, gas venting below containment liner systems, etc. However, it should be realized that the seepage quantities transmitted by in-plane flow of geotextiles (typically on the order of 2×10^{-5} m^3/s/linear meter of geotextile under a pressure equivalent to 0.6 m of soil) are relatively small when compared to the seepage capacity of 0.150 to 0.3 m of sand or other typical filter materials. Therefore, geotextiles should only replace sand or other filter layers where they can handle high seepage quantities. Remember, too, that seepage quantities are highly affected by compressive forces, incomplete saturation, and hydraulic gradients.

In recent years, special geocomposite materials have been developed which consist of cores of extruded and fluted plastics sheets, three-dimensional meshes and mats, plastic waffles, and nets and channels to convey water, which are covered by a geotextile on one or both sides to act as a filter. Geocomposite drains may be prefabricated or fabricated on site. They generally range in thickness from 5 mm to 25 mm or greater and have transmission capabilities of between 0.0002 and 0.01 m^3/s/linear width of drain. Some geocomposite systems are shown in Figure 2-9. Geocomposite drains have been used in six major areas:

1. Edge drains for pavements.
2. Interceptor trenches on slopes.
3. Drainage behind abutments and retaining structures.
4. Relief of water pressures on buried structures.
5. Substitute for conventional sand drains.
6. Waste containment systems for leachate collection and gas venting.

Prefabricated geocomposite drains are essentially used to replace or support conventional drainage systems. According to Hunt (1982), prefabricated drains offer a readily available material with known filtration and hydraulic flow properties; easy installation, and, therefore, construction economies; and protection of any waterproofing applied to the structure's exterior. Cost of prefabricated drains typically ranges from $4.50 to $25.00 per square meter. The high material cost is usually offset by expedient construction and reduction in required quantities of select granular materials. For example, geocomposites used for pavement edge drains typically cost $1.75 to $5.00/linear meter installed.

2.11-1 Design Criteria

For the geotextile design and selection with in-plane drainage capabilities and geocomposite drainage systems, there are three basic design considerations:

1. Adequate filtration without clogging or piping.
2. Adequate inflow/outflow capacity under design loads to provide maximum anticipated seepage during design life.
3. System performance considerations.

As with conventional drainage systems, geotextile selection should be based on the grain size of the material to be protected, permeability requirements, clogging resistance, and physical property requirements, as described in Section 2.3. In pavement drainage systems, dynamic loading means severe hydraulic conditions (Table 2-1). If, for example, the geotextile supplied with the geocomposite drainage system is not appropriate for your design conditions, system safety will be compromised and you should specify alternate geotextiles. This is important especially when prefabricated drains are used in critical situations and where failure system could lead to structure failure.

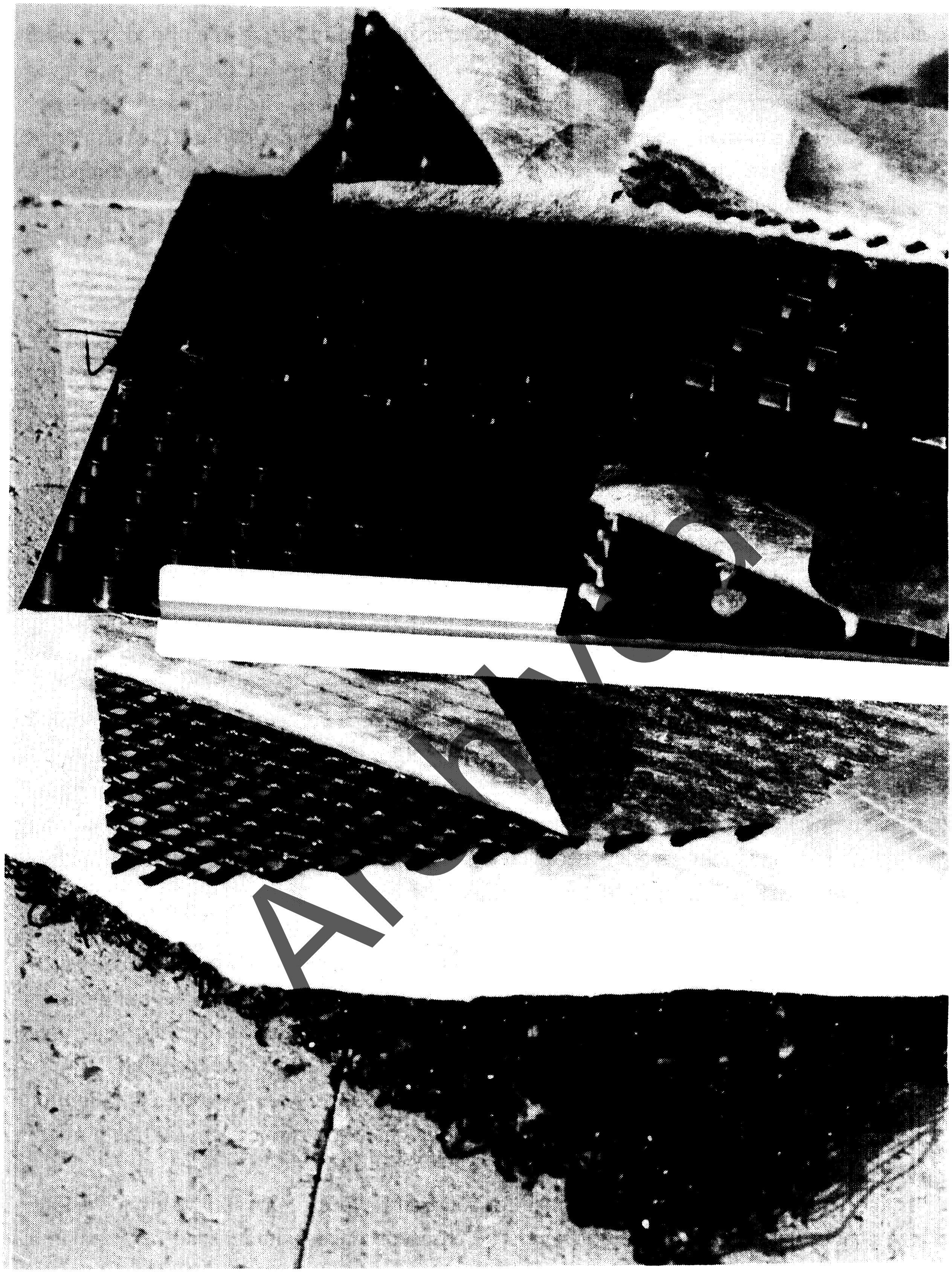


Figure 2-9 Geocomposite drains.

The maximum seepage flow into the system must be estimated and the geotextile or geocomposite selected on the basis of seepage requirements. The flow capacity of the geocomposite or geotextile can be determined from the transmissivity of the material. The test for transmissivity is ASTM D 4716, Constant Head Hydraulic Transmissivity (In-Plane Flow) of Geotextiles and Geotextile Related Products. The flow capacity per unit width of the geotextile or geocomposite can then be calculated using Darcy's Law:

$$q \ = \ k_p \, i \, A \ = \ k_p \, i \, B \, t \qquad [2 - 15]$$

or,

$$q/B \ = \ \theta \, i \qquad [2 - 16]$$

where:

q	=	flow rate (L^3/T)
k_p	=	in-plane coefficient of permeability for the geosynthetic (L/T)
B	=	width of geosynthetic (L)
t	=	thickness of geosynthetic (L)
θ	=	transmissivity of geosynthetic (= $k_p t$) (L^2/T)
i	=	hydraulic gradient (L/L)

The flow rate per unit width of the geosynthetic can then be compared with the flow rate per unit width required of the drainage system. It should be recognized that the in-plane flow capacity for geosynthetic drains reduces significantly under compression (Giroud, 1980). Additional decreases in transmissivity may occur with time due to creep. Therefore, the material should be evaluated by an appropriate laboratory model (performance) test, under the anticipated design loading conditions (with a safety factor) for the design life of the project.

Long-term compressive stress and eccentric loadings on the core of a geocomposite should be considered during design and selection. Though not yet addressed in standardized test methods or standards of practice, the following criteria (Berg, 1993) are suggested for addressing core compression. The design pressure on a geocomposite core should be limited to either:

i) the maximum pressure sustained on the core in a test of 10,000 hour minimum duration; or

ii) the crushing pressure of a core, as defined with a quick loading test, divided by a safety factor of 5.

Note that crushing pressure can only be defined for some core types. For cases where a crushing pressure cannot be defined, suitability should be based on the maximum load resulting in a residual thickness of the core adequate to provide the required flow after 10,000 hours.

Intrusion of the geotextiles into the core and long-term outflow capacity should be measured with a sustained transmissivity test (Berg, 1993). The ASTM D 4716 test procedure (1987), Constant Head Hydraulic Transmissivity of Geotextiles and Geotextile Related Products, should be followed. Test procedure should be modified for sustained testing and for use of sand substratum and super-stratum in lieu of closed cell foam rubber. Load should be maintained for 300 hours or until equilibrium is reached, whichever is greater.

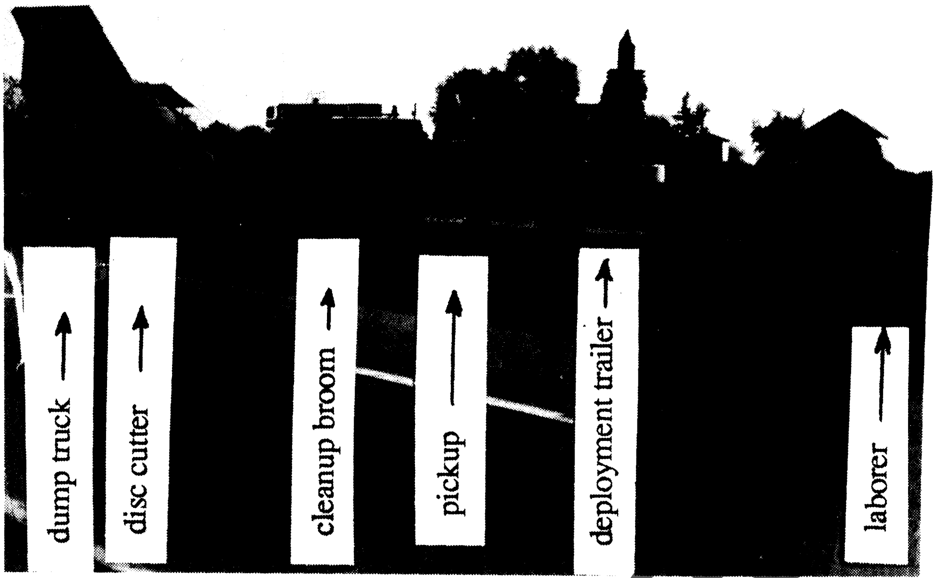
Finally, special consideration must be given to drain location and pressures on the wall when using geosynthetics to drain earth retaining structures and abutments. It is important that the drain be located away from the back of the wall and be appropriately inclined so it can intercept seepage before it impinges on the back of the wall. Placement of a thin vertical drain directly against a retaining wall may actually increase seepage forces on the wall due to rainwater infiltration (Terzaghi and Peck, 1967; and Cedergren, 1989). For further discussion of this point, see Christopher and Holtz (1985).

2.11-2 Construction Considerations

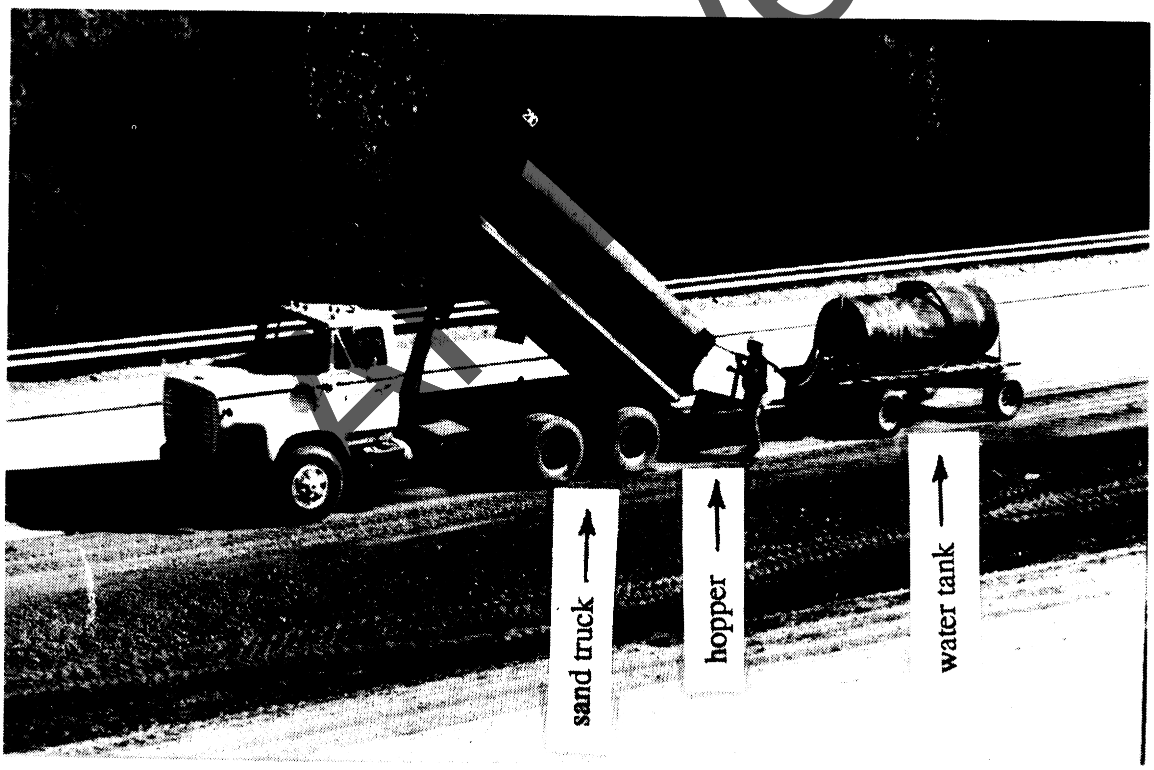
The following are considerations specific to the installation of geocomposite drains:

1. As with all geotextile applications, care should be taken during storage and placement to avoid damage to the material.
2. Placement of the backfill directly against the geotextile must be closely observed, and compaction of soil directly against the material should be avoided. Otherwise, loading during placement of backfill could damage the filter or even crush the drain. Use of clean granular backfill reduces the compaction energy requirements.
3. At the joints, where the sheets or strips of geocomposite butt together, the geotextile must be carefully overlapped to prevent soil infiltration. Also, the geotextile should extend beyond the ends of the drain to prevent soil from entering at the edges.
4. Details must be provided on how the prefabricated drains tie into the collector drainage systems.

Construction of an edge drain installation is shown in Figures 2-10 and 2-11. Additional information and recommendations regarding proper edge drain installation can be found in Koerner, et al. (1993) and in ASTM D 6088 Practice for Installation of Geocomposite Edge Drains.



(a) Equipment train used to install PGEDs according to Figure 2-11.



(b) Sand installation and backfilling equipment at end of equipment train according to Figure 2-11.

Figure 2-10 Prefabricated geocomposite edge drain construction using sand fill upstream of composite (as illustrated in Figure 2-11) (from Koerner, et al., 1993).

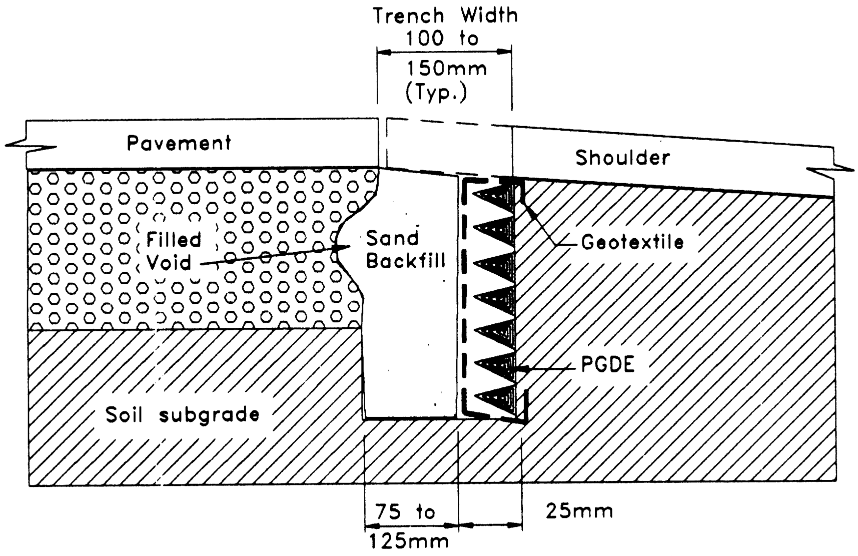


Figure 2-11 Recommended installation method for prefabricated geocomposite edge drains (from Koerner, et al., 1993).

2.12 REFERENCES

References quoted within this section are listed below. Detailed lists of specific ASTM and GRI test procedures are presented in Appendix E. A *key* reference for design is this manual (FHWA Geosynthetics Manual) and its predecessor Christopher and Holtz (1985). The recent NCHRP report (Koerner et al., 1994) specifically addresses pavement edge drain systems and is based upon analysis of failed systems. It also is a *key* reference for design. These and other *key* references are noted in bold type.

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3.0 GEOTEXTILES IN RIPRAP REVETMENTS AND OTHER PERMANENT EROSION CONTROL SYSTEMS

3.1 BACKGROUND

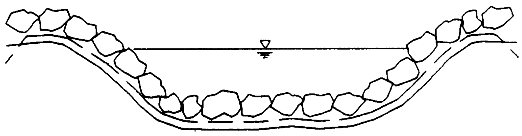
As in drainage systems, geotextiles can effectively replace graded granular filters typically used beneath riprap or other hard armor materials in revetments and other erosion control systems designed to keep soil in place. This was one of the first applications of woven monofilament geotextiles in the United States; rather extensive use started in the early 1960s. Numerous case histories have shown geotextiles to be very effective compared to riprap-only systems and equally effective as conventional graded granular filters in preventing fines from migrating through the armor system, while providing a cost savings.

Since the early developments in coastal and lake shoreline erosion control, the same design concepts and construction procedures have subsequently been applied to stream bank protection (see HEC 11, FHWA, 1989), cut and fill slope protection, protection of various small drainage structures (see HEC 14, FHWA, 1983) and ditches (see HEC 15, FHWA, 1988), wave protection for causeway and shoreline roadway embankments, and scour protection for structures such as bridge piers and abutments (see HEC 18, FHWA, 1995, and HEC 23, FHWA, 1997). Design guidelines and construction procedures for these and other similar permanent erosion control applications are presented in sections 3.3 through 3.10. Hydraulic design considerations can be found in the AASHTO *Model Drainage Manual* (1991) and the above FHWA Hydraulic Engineering Circulars. Also note that, at the time of printing of this manual, a new FHWA course and text entitled Identifying and Controlling Erosion and Sedimentation was under development.

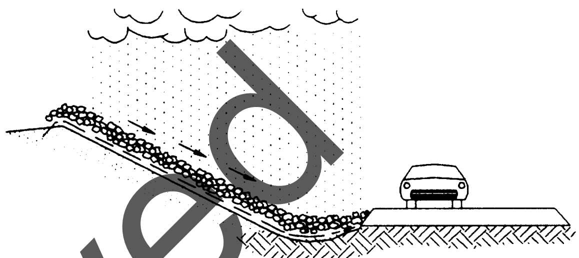
Erosion control *mats* are another type of geosynthetic used in permanent erosion control systems. They are also referred to as a Rolled Erosion Control Product (RECP). These three-dimensional mats retain soil and moisture, thus promoting vegetation growth. Vegetation roots grow through and are reinforced by the mat. The *reinforced grass* system is capable of withstanding short-term (*e.g.*, 2 hours), high velocity (*e.g.*, 6 m/s) flows with minimal erosion. Erosion control mats are addressed in section 3.11. Sediment control and temporary erosion control designed to keep soil within a prescribed boundary, including the use of geotextiles as silt fences, erosion control *blankets*, and other geosynthetics, are covered in Chapter 4.

3.2 APPLICATIONS

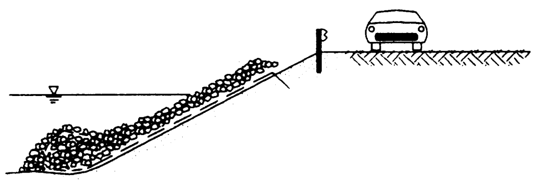
- Riprap-geotextile systems have found successful application in protecting precipitation runoff collection and high-velocity diversion ditches.



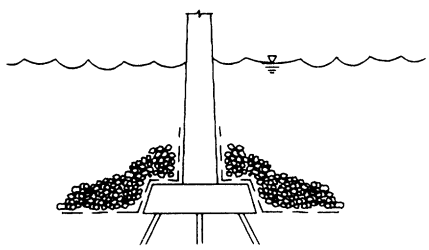
- Geotextiles may be used in slope protection to prevent or reduce erosion from precipitation, surface runoff, and internal seepage or piping. In this instance, the geotextile may replace one or more layers of granular filter materials which would be placed on the slope in conventional applications.



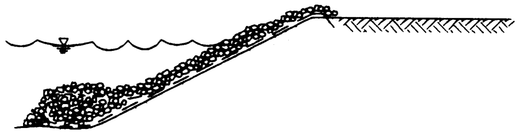
- Erosion control systems with geotextiles may also be required along streambanks to prevent encroachment of roadways or appurtenant facilities.



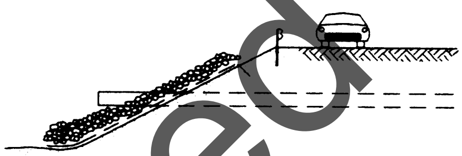
- Similarly, they may be used for scour protection around structures.



- A riprap-geotextile system can also be effective in reducing erosion caused by wave attack or tidal variations when facilities are constructed across or adjacent to large bodies of water.

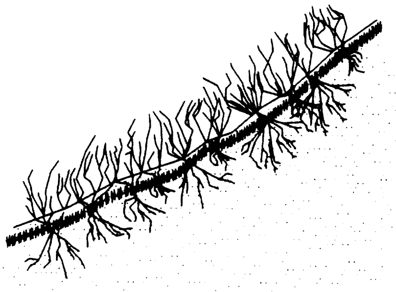


- Finally, hydraulic structures such as culverts, drop inlets, and artificial stream channels may require protection from erosion. In such applications, if vegetation cannot be established or the natural soil is highly erodible, a geotextile can be used beneath armor materials to increase erosion resistance.



In several of the above applications, placement of the filter layer may be required below water. In these cases, in comparison with conventional granular filter layers, geotextiles provide easier placement and continuity of the filter medium is assured.

- Geosynthetic erosion control *mats* are made of synthetic meshes and webbings and reinforce the vegetation root mass to provide tractive resistance to high water velocity on slopes and in ditches. These three-dimensional mats retain soil, moisture, and seed, and thus promote vegetative growth.



3.3 DESIGN OF GEOTEXTILES BENEATH HARD ARMOR

Geotextile design for hard armor erosion control systems is essentially the same as geotextile design for filters in subsurface drainage systems discussed in Section 2.3. Table 3-1 reiterates the design criteria and highlights special considerations for geotextiles beneath hard armor erosion control systems. The following is a discussion of these special considerations.

3.3-1 Retention Criteria for Cyclic or Dynamic Flow

In cyclic or dynamic flow conditions, soil particles may be able to move behind the geotextile if it is not properly weighted down. Thus, the coefficient B = 1 may not be conservative, as the bridging network (Figure 2-2) may not develop and the geotextile may be required to retain even the finer particles of soil. If there is a risk that uplift of the armor system can occur, it is recommended that the B value be reduced to 0.5 or less; that is, the largest hole in the geotextile should be small enough to retain the smaller particles of soil.

In absence of detailed design, the AASHTO M 288 Standard Specification for Geotextiles (1997) provides the following recommended maximum AOS values in relation to percent of situ soil passing the 0.075 mm sieve: (i) 0.43 mm for less than 15% passing; (ii) 0.25 mm for 15 to 50% passing; and (iii) 0.22 mm for more than 50% passing. However, for cohesive soils with a plasticity index greater than 7, the maximum AOS size is 0.30 mm. These default AOS values are based upon the predominant particle sizes of the in situ soil. The engineer may require performance testing based on engineering design for erosion control systems in problematic soil environments. Site specific testing should be performed especially if one or more of the following problematic soil environments are encountered: unstable or highly erodible soils such as non-cohesive silts; gap graded soils; alternating sand/silt laminated soils; dispersive clays; and/or rock flour.

In many erosion control applications it is common to have high hydraulic stresses induced by wave or tidal action. The geotextile may be loose when it spans between large armor stone or large joints in block-type armor systems. For these conditions, it is recommended that an intermediate layer of finer stone or gravel be placed over the geotextile and that riprap of sufficient weight be placed to prevent wave action from moving either stone or geotextile. For all applications where the geotextile can move, and when it is used as sandbags, it is recommended that samples of the site soils be washed through the geotextile to determine its particle-retention capabilities.

3.3-2 Permeability and Effective Flow Capacity Requirements for Erosion Control

In certain erosion control systems, portions of the geotextile may be covered by the armor stone or concrete block revetment systems, or the geotextile may be used to span joints in sheet pile bulkheads. For such systems, it is especially important to evaluate the flow rate required through

TABLE 3-1
SUMMARY OF GEOTEXTILE DESIGN AND SELECTION CRITERIA FOR HARD ARMOR EROSION CONTROL APPLICATIONS

I. SOIL RETENTION (PIPING RESISTANCE CRITERIA)[1]

Soils	Steady State Flow		Dynamic, Pulsating and Cyclic Flow (if geotextile can move)
<50% Passing[2] 0.075 mm	AOS or $O_{95} \leq B\ D_{85}$		
	$C_u \leq 2$ or ≥ 8:	$B = 1$	
	$2 \leq C_u \leq 4$:	$B = 0.5\ C_u$	$O_{95} \leq 0.5\ D_{85}$
	$4 \leq C_u \leq 8$:	$B = 8/C_u$	
≥50% Passing 0.075 mm	Woven: $O_{95} \leq D_{85}$		$O_{95} \leq 0.5\ D_{85}$
	Nonwoven: $O_{95} \leq 1.8\ D_{85}$		
For cohesive soils (PI > 7)	O_{95} (geotextile) ≤ 0.3 mm		

II. PERMEABILITY/PERMITTIVITY CRITERIA[3]

A. Critical/Severe Applications
$k_{geotextile} \geq 10\ k_{soil}$

B. Less Critical/Less Severe Applications (with Clean Medium to Coarse Sands and Gravels)
$k_{geotextile} \geq k_{soil}$

C. Permittivity Requirement
- $\psi \geq 0.7\ sec^{-1}$ for < 15% passing 0.075 mm
- $\psi \geq 0.2\ sec^{-1}$ for 15 to 50% passing 0.075 mm
- $\psi \geq 0.1\ sec^{-1}$ for > 50 % passing 0.075 mm

III. CLOGGING CRITERIA

A. Critical/Severe Applications[4]
Select geotextile meeting I, II, IIIB, and perform soil/geotextile filtration tests before specification, prequalifying the geotextile, or after selection before bid closing. Alternative: use approved list specification for filtration applications. Suggested performance test method: Gradient Ratio, ASTM D 5101 for cohesionless soils or Hydraulic Conductivity Ratio, ASTM D 5567 for cohesive soils.

B. Less Critical/Less Severe Applications

1. Perform soil-geotextile filtration tests.
2. Alternative: $O_{95} \geq 3\ D_{15}$ for $C_u > 3$
3. For $C_u \leq 3$, specify geotextile with maximum opening size possible from retention criteria
4. Apparent Open Area Qualifiers

For soils with % passing 0.075 mm	≥ 5%	≤ 5%
Woven monofilament geotextiles: Percent Open Area : ≥	4%	10%
Nonwoven geotextiles: Porosity[5] ≥	50%	70%

IV. SURVIVABILITY REQUIREMENTS

GEOTEXTILE STRENGTH PROPERTY REQUIREMENTS[1,2,3,4]
FOR PERMANENT EROSION CONTROL GEOTEXTILES
(after AASHTO, 1997)

Property	ASTM Test Method	Units	Geotextile Class 1[5,6] Elongation[8] < 50%	Geotextile Class 1[5,6] Elongation[8] ≥ 50%	Geotextile Class 2[5,6,7] Elongation[8] < 50%	Geotextile Class 2[5,6,7] Elongation[8] ≥ 50%
Grab Strength	D 4632	N	1400	900	1100	700
Sewn Seam Strength[9]	D 4632	N	1260	810	990	630
Tear Strength	D 4533	N	500	350	400[10]	250
Puncture Strength	D 4833	N	500	350	400	250
Burst Strength	D 3786	kPa	3500	1700	2700	1300
Ultraviolet Stability	D 4355	%	50% strength retained after 500 hours of exposure			

NOTES:
1. Acceptance of geotextile material shall be based on ASTM D 4759.
2. Acceptance shall be based upon testing of either conformance samples obtained using Procedure A of ASTM D 4354, or based on manufacturer's certifications and testing of quality assurance samples obtained using Procedure B of ASTM D 4354.
3. Minimum; use value in weaker principal direction. All numerical values represent minimum average roll value (*i.e.*, test results from any sampled roll in a lot shall meet or exceed the minimum values in the table). Lot samples according to ASTM D 4354.
4. Woven slit film geotextiles will not be allowed.
5. Use Class 2 for woven monofilament geotextiles, and Class 1 for all other geotextiles.
6. As a general guideline, the default geotextile selection is appropriate for conditions of equal or less severity than either of the following:
   a) Armor layer stone weights do not exceed 100 kg, stone drop is less than 1 m, and no aggregate bedding layer is required.
   b) Armor layer stone weights exceed 100 kg, stone drop height is less than 1 m, and the geotextile is protected by a 150 mm thick aggregate bedding layer designed to be compatible with the armor layer.
   More severe applications require an assessment of geotextile survivability based on a field trial section and may require a geotextile with higher strength properties.
7. The engineer may specify a Class 2 geotextile based on one or more of the following:
   a) The engineer has found Class 2 geotextiles to have sufficient survivability based on field experience.
   b) The engineer has found Class 2 geotextiles to have sufficient survivability based on laboratory testing and visual inspection of a geotextile sample removed from a field test section constructed under anticipated field conditions.
   c) Armor layer stone weighs less than 100 kg, stone drop height is less than 1 m, and the geotextile is protected by a 150 mm thick aggregate bedding layer designed to be compatible with the armor layer.
   d) Armor layer stone weights do not exceed 100 kg, stone is placed with a zero drop height.
8. As measured in accordance with ASTM D 4632.
9. When seams are required. Values apply to both field and manufactured seams.
10. The required MARV tear strength for woven monofilament geotextiles is 250 N.

the open portion of the system and select a geotextile that meets those flow requirements. Again, since flow is restricted through the geotextile, the required flow capacity is based on the flow capacity of the area available for flow; or

$$q_{required} = q_{geotextile}(A_g/A_t) \qquad \text{(Eq. 2 - 9)}$$

where: A_g = geotextile area available for flow, and
A_t = total geotextile area.

The AASHTO M 288 Standard Specification for Geotextiles (1997) presents recommended minimum permittivity values in relation to percent of situ soil passing the 0.075 mm sieve. The values are the same as presented in Table 3-1. The default permittivity values are based upon the predominant particle sizes of the in situ soil. Again, the engineer may require performance testing based on engineering design for drainage systems in problematic soil environments.

3.3-3 Clogging Resistance for Cyclic or Dynamic Flow

Since erosion control systems are often used on highly erodible soils with reversing and cyclic flow conditions, severe hydraulic conditions often exist. Accordingly, designs should reflect these conditions, and soil-geotextile filtration tests should always be conducted. Since these tests are performance-type tests and require project site soil samples, they must be conducted by the owner or an owner representative and not by the geotextile manufacturers or suppliers. For sandy and silty soils ($k \geq 10^{-7}$ m/s) the long-term, gradient ratio test (ASTM D 5101) is recommended as described in Chapter 1. For fine-grained soils, the hydraulic conductivity ratio (HCR) test (ASTM D 5567) should be considered with the modifications and caveats recommended in Chapter 1. Other filtration tests, some of which are appropriate for finer soils, are described by Christopher and Holtz (1985) and Koerner (1990), among others.

3.3-4 Survivability Criteria for Erosion Control

Because the construction procedures for erosion control systems are different than those for drainage systems, the geotextile property requirements for survivability in Table 3-1 differ somewhat from those discussed in Section 2.3-4. As placement of armor stone is generally more severe than placement of drainage aggregate, required property values are higher for each category of geotextile.

Riprap or armor stone should be large enough to withstand wave action and thus not abrade the geotextile. The specific site conditions should be reviewed, and if such movement cannot be avoided, then an abrasion requirement based on ASTM D 4886 (modified flex stoll) should be included in the specifications. Allowable physical property reduction due to abrasion should be specified. No reduction in piping resistance, permeability, or clogging resistance should be allowed after exposure to abrasion.

It is important to realize that these **minimum** survivability values are not based on any systematic research but on the properties of existing geotextiles which are known to have performed satisfactorily in hard armor erosion control applications. The values are meant to serve as guidelines for inexperienced users in selecting geotextiles for routine projects. They are not intended to replace site-specific evaluation, testing, and design.

3.4 GEOTEXTILE DESIGN GUIDELINES

STEP 1. Application evaluation.

A. Critical/less critical

1. If the erosion control system fails, will there be a risk of loss of life?
2. Does the erosion control system protect a significant structure,. and will failure lead to significant structural damage?
3. If the geotextile clogs, will failure occur with no warning? Will failure be catastrophic?
4. If the erosion control system fails, will the repair costs greatly exceed installation costs?

B. Severe/less severe

1. Are soils to be protected gap-graded, pipable, or dispersive?
2. Are soils present which consist primarily of silts and uniform sands with 85% passing the 0.15 mm sieve?
3. Will the erosion control system be subjected to reversing or cyclic flow conditions such as wave action or tidal variations?
4. Will high hydraulic gradients exist in the soils to be protected? Will rapid drawdown conditions or seeps or weeps in the soil exist? Will blockage of seeps and weeps produce high hydraulic pressures?
5. Will high-velocity conditions exist, such as in stream channels?

NOTE: If the answer is yes to any of the above questions, the design should proceed under the *critical/severe* requirements; otherwise use the *less critical/less severe* design approach.

STEP 2. Obtain soil samples from the site.

A. Perform grain size analyses

1. Determine percent passing the 0.075 mm sieve.

2. Determine the plastic index (PI).
3. Calculate $C_u = D_{60}/D_{10}$.

NOTE: When the protected soil contains particles passing the 0.075 mm sieve, use only the gradation of soil passing the 4.75 mm sieve in selecting the geotextile (*i.e.*, scalp off the +4.75 mm material).

4. Obtain D_{85} for each soil and select the worst case soil (*i.e.*, soil with smallest B x D_{85}) for retention.

B. Perform field or laboratory permeability tests

1. Select worse case soil (*i.e.*, soil with highest coefficient of permeability k).

NOTE: The permeability of clean sands (<5% passing 0.075 mm sieve) with 0.1 mm D_{10} < 3 mm and C_u < 5 can be estimated by Hazen's formula, $k = (D_{10})^2$ (k in cm/s; D_{10} in mm). This formula should not be used for finer-grained soils.

STEP 3. Evaluate armor material and placement.
Design reference: FHWA Hydraulic Engineering Circular No. 15 (FHWA, 1988).

A. Size armor stone or riprap

Where minimum size of stone exceeds 100 mm, or greater than a 100 mm gap exists between blocks, an intermediate gravel layer 150 mm thick should be used between the armor stone and geotextile. Gravel should be sized such that it will not wash through the armor stone (*i.e.*, D_{85} gravel $\geq D_{15}$ riprap/5).

B. Determine armor stone placement technique (*i.e.*, maximum height of drop).

STEP 4. Calculate anticipated reverse flow through erosion control system.

Here we need to estimate the maximum flow from seeps and weeps, maximum flow from wave runout, or maximum flow from rapid drawdown.

A. General case -- use Darcy's law

$$q = kiA \qquad \text{(Eq. 2 - 15)}$$

where:

q = outflow rate (L^3/T)
k = effective permeability of soil (from Step 2B above) (L/T)
i = average hydraulic gradient in soil (*e.g.*, tangent of slope angle for wave runoff)(dimensionless)

A = area of soil and drain material normal to the direction of flow (L^2). Can be evaluated using a unit area.

Use a conventional flow net analysis (Cedergren, 1977) for seepage through dikes and dams or from a rapid drawdown analysis.

B. Specific erosion control systems -- Hydraulic characteristics depend on expected precipitation, runoff volumes and flow rates, stream flow volumes and water level fluctuations, normal and maximum wave heights anticipated, direction of waves and tidal variations. Detailed information on determination of these parameters is available in the FHWA (1989) Hydraulic Engineering Circular No. 11.

STEP 5. Determine geotextile requirements.

A. Retention Criteria

From Step 2A, obtain D_{85} and C_u; then determine largest pore size allowed.

AOS or $O_{95(geotextile)} < B\ D_{85(soil)}$ (Eq. 2 - 1)

where: B = 1 for a conservative design.

For a less-conservative design and for ≤ 50% passing 0.075 mm sieve:

$B = 1$ for $C_u \leq 2$ or ≥ 8 (Eq. 2 - 2a)

$B = 0.5\ C_u$ for $2 \leq C_u \leq 4$ (Eq. 2 - 2b)

$B = 8/C_u$ for $4 < C_u < 8$ (Eq. 2 - 2c)

For ≥ 50% passing 0.075 mm sieve:

$B = 1$ for wovens

$B = 1.8$ for nonwovens

and AOS or O_{95} (geotextile) ≤ 0.3 mm (Eq. 2 - 5)

For nondispersive cohesive soils (PI > 7) use:

AOS or $O_{95} \leq 0.3$ mm

If geotextile and soil retained by it can move:

$B = 0.5$

B. Permeability/Permittivity Criteria

1. Less Critical/Less Severe

$k_{geotextile} \geq k_{soil}$ (Eq. 2 - 7a)

2. Critical/Severe

$k_{geotextile} \geq 10\ k_{soil}$ (Eq. 2 - 8a)

3. Permittivity ψ Requirement

$\psi \geq 0.7\ sec^{-1}$ for < 15% passing 0.075 mm [3 - 1a]

$\psi \geq 0.2\ sec^{-1}$ for 15 to 50% passing 0.075 mm [3 - 1b]

$\psi \geq 0.1\ sec^{-1}$ for > 50% passing 0.075 mm [3 - 1c]

4. Flow Capacity Requirement

$q_{geotextile} \geq (A_t/A_g)\ q_{required}$ (from Eq. 2 - 9)

or

$(k_{geotextile}/t)\ h\ A_g \geq q_{required}$

where: $q_{required}$ is obtained from Step 4 (Eq. 15) above.

$k_{geotextile}/t = \psi$ = permittivity

h = average head in field

A_g = area of fabric available for flow (*e.g.*, if 50% of geotextile covered by flat rocks or riprap, A_g = 0.5 total area)

A_t = total area of geotextile

C. Clogging Criteria

1. Less critical/less severe

a. Perform soil-geotextile filtration tests.

b. Alternative: From Step 2A obtain D_{15}; then determine minimum pore size requirement, for soils with C_u > 3, from

$O_{95} \geq 3\ D_{15}$ (Eq. 2 - 10)

c. Other qualifiers

For soils with % passing 0.075 mm	$\geq$ 5%	$\leq$5%
Woven monofilament geotextiles: Percent Open Area $\geq$	4%	10%
Nonwoven geotextiles: Porosity $\geq$	50%	70%

2. Critical/severe

Select geotextiles that meet retention, permeability, and survivability criteria; as well as the criteria in Step 5C.1 above; perform a filtration test.

Suggested filtration test for sandy and silty soils (*i.e.*, k > 10^{-7} m/s) is the gradient ratio test as described in Chapter 1. The hydraulic conductivity ratio test (see Chapter 1) is recommended for fine-grained soils (*i.e.*, k < 10^{-7} m/s), if appropriately modified.

D. Survivability

Select geotextile properties required for survivability from Table 3-1. Add durability requirements if applicable. Don't forget to check for abrasion and check drop height. Evaluate worst case scenario for drop height.

STEP 6. Estimate costs.

Calculate the volume of armor stone, the volume of aggregate and the area of the geotextile. Apply appropriate unit cost values.

Grading and site preparation (LS)	________
Geotextile (/m^2)	________
Geotextile placement (/m^2)	________
In-place aggregate bedding layer (/m^2)	________
Armor stone (/kg)	________
Armor stone placement (/kg)	________
Total cost	________

STEP 7. Prepare specifications.

Include for the geotextile:

A. General requirements
B. Specific geotextile properties
C. Seams and overlaps
D. Placement procedures
E. Repairs
F. Testing and placement observation requirements

See Sections 1.6 and 3.7 for specification details.

STEP 8. Obtain samples of the geotextile before acceptance.

STEP 9. Monitor installation during construction, and control drop height. Observe erosion control systems during and after significant storm events.

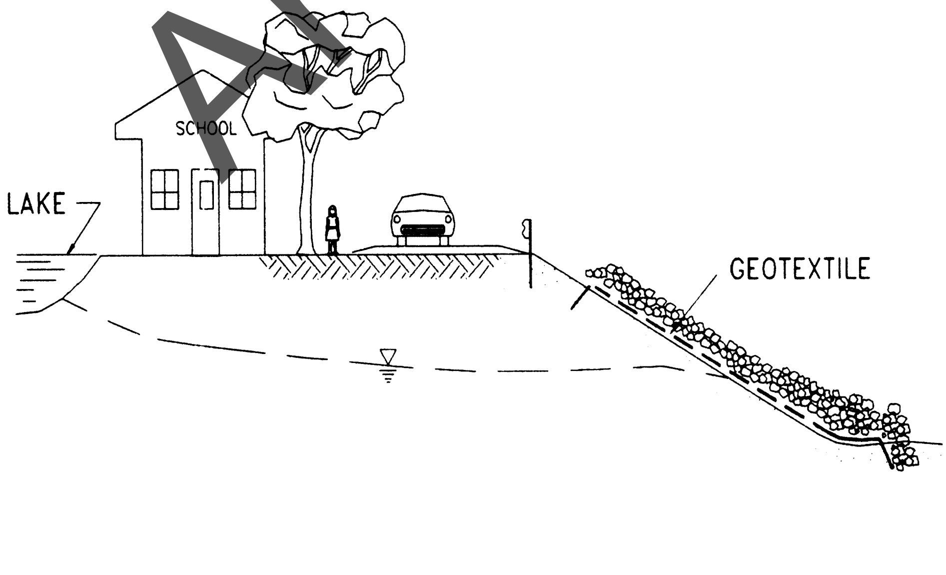
3.5 GEOTEXTILE DESIGN EXAMPLE

DEFINITION OF DESIGN EXAMPLE

- Project Description: Riprap on slope is required to permit groundwater seepage out of slope face, without erosion of slope. See figure for project cross section.
- Type of Structure: small stone riprap slope protection
- Type of Application: geotextile filter beneath riprap
- Alternatives: i) graded soil filter; or
  ii) geotextile filter between embankment and riprap

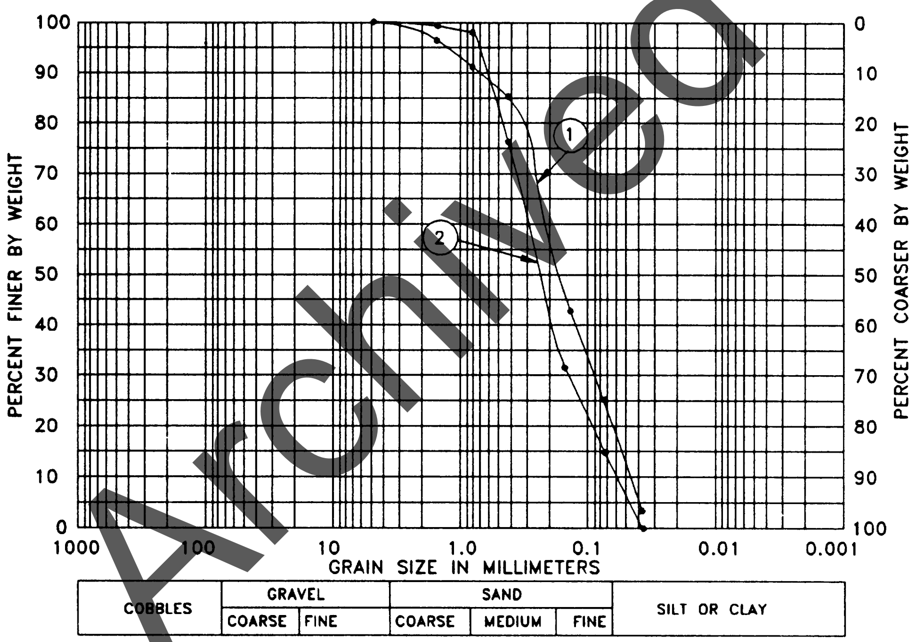
GIVEN DATA

- see cross section
- riprap is to allow unimpeded seepage out of slope
- riprap will consist of small stone (50 to 300 mm)
- stone will be placed by dropping from a backhoe
- seeps have been observed in the existing slope
- soil beneath the proposed riprap is a fine silty sand
- gradations of two *representative* soil samples



Project Cross Section

SIEVE SIZE (mm)	PERCENT PASSING, BY WEIGHT	
	Sample A	Sample B
4.75	100	100
1.68	96	100
0.84	92	98
0.42	85	76
0.15	43	32
0.075	25	15
0.037	3	0



Grain Size Distribution Curve

DEFINE

A. Geotextile function(s)

B. Geotextile properties required

C. Geotextile specification

SOLUTION

A. Geotextile function(s):

Primary	-	filtration
Secondary	-	separation

B. Geotextile properties required:
- apparent opening size (AOS)
- permittivity
- survivability

DESIGN

STEP 1. EVALUATE CRITICAL NATURE AND SITE CONDITIONS.

From given data, this is a critical application due to potential for loss of life and potential for significant structural damage.
Soils are well-graded, hydraulic gradient is low for this type of application, and flow conditions are steady state.

STEP 2. OBTAIN SOIL SAMPLES.

A. GRAIN SIZE ANALYSES
Plot gradations of representative soils. The D_{60}, D_{10}, and D_{85} sizes from the gradation plot are noted in the table below for Samples A and B.

Soil Sample	$D_{60} \div D_{10} = C_u$	B =	$B \times D_{85} \geq$ AOS (mm)
A	$0.20 \div 0.045 = 4.4$	$8 \div C_u = 8 \div 4.4 = 1.82$	$1.82 \times 0.44 = 0.8$
B	$0.30 \div 0.06 = 5$	$8 \div C_u = 8 \div 5 = 1.6$	$1.6 \times 0.54 = 0.86$

Worst case soil for retention is Soil A, with D_{85} equal to 0.44 mm.

B. PERMEABILITY TESTS
This is a critical application and soil permeability tests should be conducted. An estimated permeability will be used for preliminary design purposes.

STEP 3. EVALUATE ARMOR MATERIAL AND PLACEMENT.

A. Small stone (50 to 300 mm) riprap will be used.

B. A placement drop of less than 1 m will be specified.

STEP 4. CALCULATE ANTICIPATED FLOW THROUGH SYSTEM.
Flow computations are not included within this example. The entire height of the slope face will be protected, to add to conservatism of design.

STEP 5. DETERMINE GEOTEXTILE REQUIREMENTS.

A. RETENTION

$$AOS < B\, D_{85} \qquad \text{(Eq. 2 - 1)}$$

Determine uniformity coefficient, C_u, coefficient B, and the maximum AOS.
Sample A controls (see table above), therefore, **AOS ≤ 0.8 mm**

B. PERMEABILITY/PERMITTIVITY

This is a critical application, therefore,

$$k_{geotextile} \geq 10\, x\, k_{soil}$$

Estimate permeability (after Hazen's formula, which is for clean sands), for preliminary design,

$$k \approx (D_{10})^2$$

where: k = approximate soil permeability (cm/sec); and
D_{10} is in mm.

k_{soil} = $2.0\,(10)^{-3}$ cm/sec for Sample A
$3.6\,(10)^{-3}$ cm/sec for Sample B

Therefore (with rounding the number), **$k_{geotextile} \geq 4\,(10)^{-2}$ cm/sec**

Since 15% to 25% of the soil to be protected is finer than 0.075 mm, from Table 3-1:

$\psi_{geotextile} \geq 0.2\ sec^{-1}$

C. CLOGGING

As the project is critical, a filtration test is recommended to evaluate clogging potential. Select geotextile(s) meeting retention, permeability, survivability criteria, and the following qualifiers. Run filtration test (*e.g.*, gradient ratio) and prequalify materials or test representative materials to confirm compatibility.

Minimum Opening Size Qualifier (for $C_u > 3$): $O_{95} \geq 3\, D_{15}$

$O_{95} \geq$ $3\, x\, 0.057 = 0.17$ mm for Sample A
$3\, x\, 0.079 = 0.24$ mm for Sample B

Sample A controls, therefore, $O_{95} \geq$ **0.17 mm**

Other Qualifiers, since greater than 5% of the soil to be protected is finer than 0.075 mm, from Table 3-1:

for Nonwovens - Porosity > 50 %
for Wovens - POA (Percent Open Area) > 4 %

D. SURVIVABILITY

A *Class 1* geotextile will be specified because this a critical application. Effect on project cost is minor. Therefore, from Table 3-1, the following **minimum** values will be specified:

	<u><50% Elongation</u>	<u>>50% Elongation</u>
Grab Strength	1400 N	900 N
Sewn Seam Strength	1260 N	810 N
Tear Strength	500 N	350 N
Puncture Strength	500 N	350 N
Burst Strength	3500 N	1700 N
Ultraviolet Degradation	50 % strength retained at 500 hours	

Complete Steps 6 through 9 to finish design.

STEP 6. ESTIMATE COSTS.

STEP 7. PREPARE SPECIFICATIONS.

STEP 8. COLLECT SAMPLES.

STEP 9. MONITOR INSTALLATION, AND DURING & AFTER STORM EVENTS.

3.6 GEOTEXTILE COST CONSIDERATIONS

The total cost of a riprap-geotextile revetment system will depend on the actual application and type of revetment selected. The following items should be considered:

1. grading and site preparation;
2. cost of geotextile, including cost of overlapping and pins versus cost of sewn seams;
3. cost of placing geotextile, including special considerations for below-water placement;
4. bedding materials, if required, including placement;
5. armor stone, concrete blocks, sand bags, etc.; and
6. placement of armor stone (dropped versus hand- or machine-placed).

For Item No. 2, cost of overlapping includes the extra material required for the overlap, cost of pins, and labor considerations versus the cost of field and/or factory seaming, plus the additional cost of laboratory seam testing. These costs can be obtained from manufacturers, but typical costs of a sewn seam are equivalent to 1 to 1.5 m^2 of geotextile. Alternatively, the contractor can be required to supply the cost on an area covered or *in-place* basis. For example, current U.S. Army Corps of Engineers Specifications CW-02215 (1977) require measurement for payment for geotextiles in streambank and slope protection to be on an *in-place* basis without allowance for any material in laps and seams. Further, the unit price includes furnishing all plant, labor, material, equipment, securing pins, etc., and performing all operations in connection with placement of the geotextile, including prior preparation of banks and slopes. Of course, field performance should also be considered, and sewn seams are generally preferred to overlaps.

Items 2, 4, and 6 can be compared with respect to using Moderate Survivability versus High Survivability (Table 3-1, Section IV) geotextiles based on the cost of bedding materials and placement of armor stone.

To determine cost effectiveness, benefit-cost ratios should be compared for the riprap-geotextile system versus conventional riprap-granular filter systems or other available alternatives of equal technical feasibility and operational practicality. Average cost of geotextile protection systems placed above the water level, including slope preparation, geotextile cost of seaming or securing pins, and placement is approximately $3.00-6.00 per square meter, excluding the armor stone. Cost of placement below water level can vary considerably depending on the site conditions and the contractor's experience. For below-water placement, it is recommended that prebid meetings be held with potential contractors to explore ideas for placement and discuss anticipated costs.

3.7 GEOTEXTILE SPECIFICATIONS

In addition to the general recommendations concerning specifications in Chapter 1, erosion control specifications must include construction details (see Section 3.8), as the appropriate geotextile will depend on the placement technique. In addition, the specifications should require the contractor to demonstrate through trial sections that the proposed riprap placement technique will not damage the geotextile.

Many erosion control projects may be better-served by performance-type filtration tests that provide an indication of long-term performance. Thus, in many cases, *approved list*-type specifications, as discussed in Section 1.6, may be appropriate. To develop the list of approved geotextiles, filtration studies (as suggested in Section 3.4, Step 4) should be performed using problem soils and conditions that exist in the localities where geotextiles will be used. An approved list for each condition should be established. In addition, geotextiles should be classified as High or Moderate Survivability geotextiles, in accordance with the index properties listed in Table 3-1 and construction conditions.

The following example specification is a combination of the AASHTO M288 (1997) geotextile material specification and its accompanying construction/installation guidelines. It includes the requirements discussed in Section 1.6 for a good specification. As with the specification presented in Chapter 2, site-specific hydraulic and physical properties must be appropriately selected and included.

EROSION CONTROL GEOTEXTILE SPECIFICATION
(after AASHTO M288, 1997)

1. SCOPE

1.1 *Description.* This specification is applicable to the use of a geotextile between energy absorbing armor systems and the in situ soil to prevent soil loss resulting in excessive scour and to prevent hydraulic uplift pressure causing instability of the permanent erosion control system. This specification does not apply to other types of geosynthetic soil erosion control materials such as turf reinforcement mats.

2. REFERENCED DOCUMENTS

2.1 *AASHTO Standards*

T88 Particle Size Analysis of Soils
T90 Determining the Plastic Limit and Plasticity Index of Soils
T99 The Moisture-Density Relationships of Soils Using a 2.5 kg Rammer and a 305 mm Drop

2.2 *ASTM Standards*

D 123 Standard Terminology Relating to Textiles
D 276 Test Methods for Identification of Fibers in Textiles
D 3786 Test Method for Hydraulic Burst Strength of Knitted Goods and Nonwoven Fabrics, Diaphragm Bursting Strength Tester Method
D 4354 Practice for Sampling of Geosynthetics for Testing
D 4355 Test Method for Deterioration of Geotextiles from Exposure to Ultraviolet Light and Water (Xenon Arc Type Apparatus)
D 4439 Terminology for Geosynthetics
D 4491 Test Methods for Water Permeability of Geotextiles by Permittivity
D 4632 Test Method for Grab Breaking Load and Elongation of Geotextiles
D 4751 Test Method for Determining Apparent Opening Size of a Geotextile
D 4759 Practice for Determining the Specification Conformance of Geosynthetics
D 4833 Test Method for Index Puncture Resistance of Geotextiles, Geomembranes and Related Products
D 4873 Guide for Identification, Storage, and Handling of Geotextiles
D 5141 Test Method to Determine Filtering Efficiency and Flow Rate for Silt Fence Applications Using Site Specific Soil

3. PHYSICAL AND CHEMICAL REQUIREMENTS

3.1 Fibers used in the manufacture of geotextiles and the threads used in joining geotextiles by sewing, shall consist of long chain synthetic polymers, composed of at least 95% by weight polyolefins or polyesters. They shall be formed into a stable network such that the filaments or yarns retain their dimensional stability relative to each other, including selvages.

3.2 *Geotextile Requirements.* The geotextile shall meet the requirements of following Table. Woven slit film geotextiles (*i.e.*, geotextiles made from yarns of a flat, tape-like character) will not be allowed. All numeric values in the following table, except AOS, represent minimum average roll values (MARV) in the weakest principal direction (*i.e.*, average test results of any roll in a lot sampled for conformance or quality assurance testing shall meet or exceed the minimum values). Values for AOS represent maximum average roll values.

> NOTE: The property values in the following table represent default values which provide for sufficient geotextile survivability under most conditions. Minimum property requirements may be reduced when sufficient survivability information is available [see Note 5 of Table 2-2 and Appendix D]. The Engineer may also specify properties different from those listed in the following Table based on engineering design and experience.

4. CERTIFICATION

4.1 The Contractor shall provide to the Engineer a certificate stating the name of the manufacturer, product name, style number, chemical composition of the filaments or yarns and other pertinent information to fully describe the geotextile.

4.2 The Manufacturer is responsible for establishing and maintaining a quality control program to assure compliance with the requirements of the specification. Documentation describing the quality control program shall be made available upon request.

Permanent Erosion Control Geotextile Requirements

Property	ASTM Test Method	Units	Geotextile		
			Woven Monofilament	All other geotextiles	
				Elongation < 50% [1]	Elongation ≥ 50% [1]
Grab Strength	D 4632	N	1100	1400	900
Sewn Seam Strength [2]	D 4632	N	990	1200	810
Tear Strength	D 4533	N	250	500	350
Puncture Strength	D 4833	N	400	500	350
Burst Strength	D 3786	kPa	2700	3500	1700
			Percent In Situ Passing 0.075 mm Sieve [3]		
			< 15	15 to 50	> 50
Permittivity	D 4491	sec^{-1}	0.7	0.2	0.1
Apparent Opening Size	D 4751	mm	0.43	0.25	0.1
Ultraviolet Stability	D 4355	%	50% after 500 hours of exposure		

NOTES:
(1) As measured in accordance with ASTM D 4632.
(2) When sewn seams are required.
(3) Based on grain size analysis of in situ soil in accordance with AASHTO T88.

4.3 The Manufacturer's certificate shall state that the furnished geotextile meets MARV requirements of the specification as evaluated under the Manufacturer's quality control program. The certificate shall be attested to be a person having legal authority to bind the Manufacturer.

4.4 Either mislabeling or misrepresentation of materials shall be reason to reject those geotextile products.

5. SAMPLING, TESTING, AND ACCEPTANCE

5.1 Geotextiles shall be subject to sampling and testing to verify conformance with this specification. Sampling for testing shall be in accordance with ASTM D 4354. Acceptance shall be based on testing of either conformance samples obtained using Procedure A of ASTM D 4354, or based on manufacturer's certifications and testing of quality assurance samples obtained using Procedure B of ASTM D 4354. A lot size for conformance or quality assurance sampling shall be considered to be the shipment quantity of the given product or a truckload of the given product, whichever is smaller.

5.2 Testing shall be performed in accordance with the methods referenced in this specification for the indicated application. The number of specimens to test per sample is specified by each test method. Geotextile product acceptance shall be based on ASTM D 4759. Product acceptance is determined by comparing the average test results of all specimens within a given sample to the specification MARV. Refer to ASTM D 4759 for more details regarding geotextile acceptance procedures.

6. SHIPMENT AND STORAGE

6.1 Geotextile labeling, shipment, and storage shall follow ASTM D 4873. Product labels shall clearly show the manufacturer or supplier name, style number, and roll number. Each shipping document shall include a notation certifying that the material is in accordance with the manufacturer's certificate.

6.2 Each geotextile roll shall be wrapped with a material that will protect the geotextile from damage due to shipment, water, sunlight, and contaminants. The protective wrapping shall be maintained during periods of shipment and storage.

6.3 During storage, geotextile rolls shall be elevated off the ground and adequately covered to protect them from the following: site construction damage, precipitation, extended ultraviolet radiation including sunlight, chemicals that are strong acids or strong bases, flames including welding sparks, temperatures in excess of 71°C (160°F), and any other environmental condition that may damage the physical property values of the geotextile.

7. CONSTRUCTION

7.1 *General.* Atmospheric exposure of geotextiles to the elements following lay down shall be a maximum of 14 days to minimize damage potential.

7.2 *Seaming.*

a. If a sewn seam is to be used for the seaming of the geotextile, the thread used shall consist of high strength polypropylene, or polyester. Nylon thread shall not be used. For erosion control applications, the thread shall also be resistant to ultraviolet radiation. The thread shall be of contrasting color to that of the geotextile itself.

b. For seams which are sewn in the field, the Contractor shall provide at least a 2 m length of sewn seam for sampling by the Engineer before the geotextile is installed. For seams which are sewn in the factory, the Engineer shall obtain samples of the factory seams at random from any roll of geotextile which is to be used on the project.

b.1 For seams that are field sewn, the seams sewn for sampling shall be sewn using the same equipment and procedures as will be used for the production of seams. If seams are to be sewn in both the machine and cross machine directions, samples of seams from both directions shall be provided.

b.2 The seam assembly description shall be submitted by the Contractor along with the sample of the seam. The description shall include the seam type, stitch type, sewing thread, and stitch density.

7.3 *Geotextile Placement.*

a. The geotextile shall be placed in intimate contact with the soils without wrinkles or folds and anchored on a smooth graded surface approved by the Engineer. The geotextile shall be placed in such a manner that placement of the overlying materials will not excessively stretch so as to tear the geotextile. Anchoring of the terminal ends of the geotextile shall be accomplished through the use of key trenches or aprons at the crest and toe of slope. See Figures 3-2 and 3-3 [this manual].

NOTE 1: In certain applications to expedite construction, 450 mm anchoring pins placed on 600 to 1800 mm centers, depending on the slope of the covered area, have been used successfully.

a.2 Care shall be taken during installation so as to avoid damage occurring to the geotextile as a result of the installation process. Should the geotextile be damaged during installation, a geotextile patch shall be placed over the damaged area extending 1 m beyond the perimeter of the damage.

b. *Armor.* The armor system placement shall begin at the toe and proceed up the slope. Placement shall take place so as to avoid stretching resulting in tearing of the geotextile. Riprap and heavy stone filling shall not be dropped from a height of more than 300 mm. Stone weighing more than 450 N shall not be allowed to roll down the slope.

b.1 Slope protection and smaller sizes of stone filling shall not be dropped from a height exceeding 1 m, or a demonstration provided showing that the placement procedures will not damage the geotextile. In under water applications, the geotextile and backfill material shall be placed the same day. All void spaces in the armor stone shall be backfilled with small stone to ensure full coverage.

b.2 Following placement of the armor stone, grading of the slope shall not be permitted if the grading results in movement of the stone directly above the geotextile.

c. *Damage.* Field monitoring shall be performed to verify that the armor system placement does not damage the geotextile.

c.1 Any geotextile damaged during backfill placement shall be replaced as directed by the Engineer, at the Contractor's expense.

8. METHOD OF MEASUREMENT

8.1 The geotextile shall be measured by the number of square meters computed from the payment lines shown on the plans or from payment lines established in writing by the Engineer. This excludes seam overlaps, but shall include geotextiles used in crest and toe of slope treatments.

8.2 Slope preparation, excavation and backfill, bedding, and cover material are separate pay items.

9. BASIS OF PAYMENT

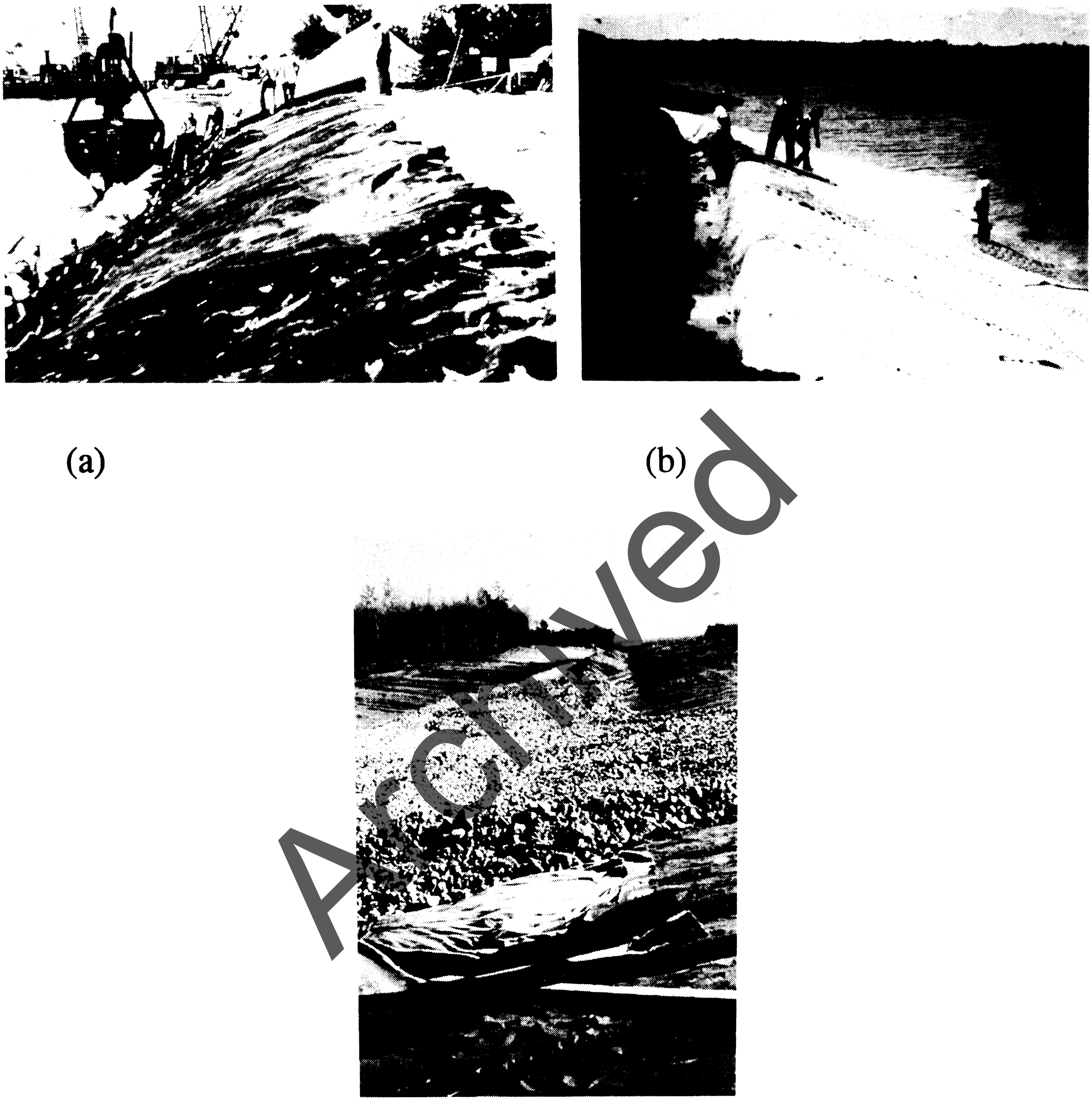
9.1 The accepted quantities of geotextile shall be paid for per square meter in place.

9.2 Payment will be made under:

Pay Item	Pay Unit
Erosion Control Geotextile	Square Meter

3.8 GEOTEXTILE INSTALLATION PROCEDURES

Construction requirements will depend on specific application and site conditions. Photographs of several installations are shown in Figure 3-1. The following general construction considerations apply for most riprap-geotextile erosion protection systems. Special considerations related to specific applications and alternate riprap designs will follow.



(a) (b)

(c)

Figure 3-1 Erosion control installations: a) installation in wave protection revetment; b) shoreline application; and c) drainage ditch application.

3.8-1 General Construction Considerations

1. Grade area and remove debris to provide smooth, fairly even surface.
   a. Depressions or holes in the slope should be filled to avoid geotextile bridging and possible tearing when cover materials are placed.
   b. Large stones, limbs, and other debris should be removed prior to placement to prevent fabric damage from tearing or puncturing during stone placement.

2. Place geotextile loosely, laid with machine direction in the direction of anticipated water flow or movement.

3. Seam or overlap the geotextile as required.
   a. For overlaps, adjacent rolls of geotextile should be overlapped a minimum of 0.3 m. Overlaps should be in the direction of water flow and stapled or pinned to hold the overlap in place during placement of stone. Steel pins are normally 5 mm diameter, 0.5 m long, pointed at one end, and fitted with 40 mm diameter washers at the other end. Pins should be spaced along all overlap alignments at a distance of approximately 1 m center to center.
   b. The geotextile should be pinned loosely so it can easily conform to the ground surface and *give* when stone is placed.
   c. If seamed, seam strength should equal or exceed the minimum seam requirements indicated in the specification section of Chapter 1.

4. The maximum allowable slope on which a riprap-geotextile system can be placed is equal to the lowest soil-geotextile friction angle for the natural ground or stone-geotextile friction angle for cover (armor) materials. Additional reductions in slope may be necessary due to hydraulic considerations and possible long-term stability conditions. For slopes greater than 2.5 to 1, special construction procedures will be required, including toe berms to provide a buttress against slippage, loose placement of geotextile sufficient to allow for downslope movement, elimination of pins at overlaps, increase in overlap requirements, and possible benching of the slope. Care should be taken not to put irregular wrinkles in the geotextile because erosion channels can form beneath the geotextile.

5. For streambank and wave action applications, the geotextile must be keyed in at the bottom of the slope. If the riprap-geotextile system cannot be extended a few meters above the anticipated maximum high water level, the geotextile should also be keyed in at the crest of the slope. Alternative key details are shown in Figure 3-2.

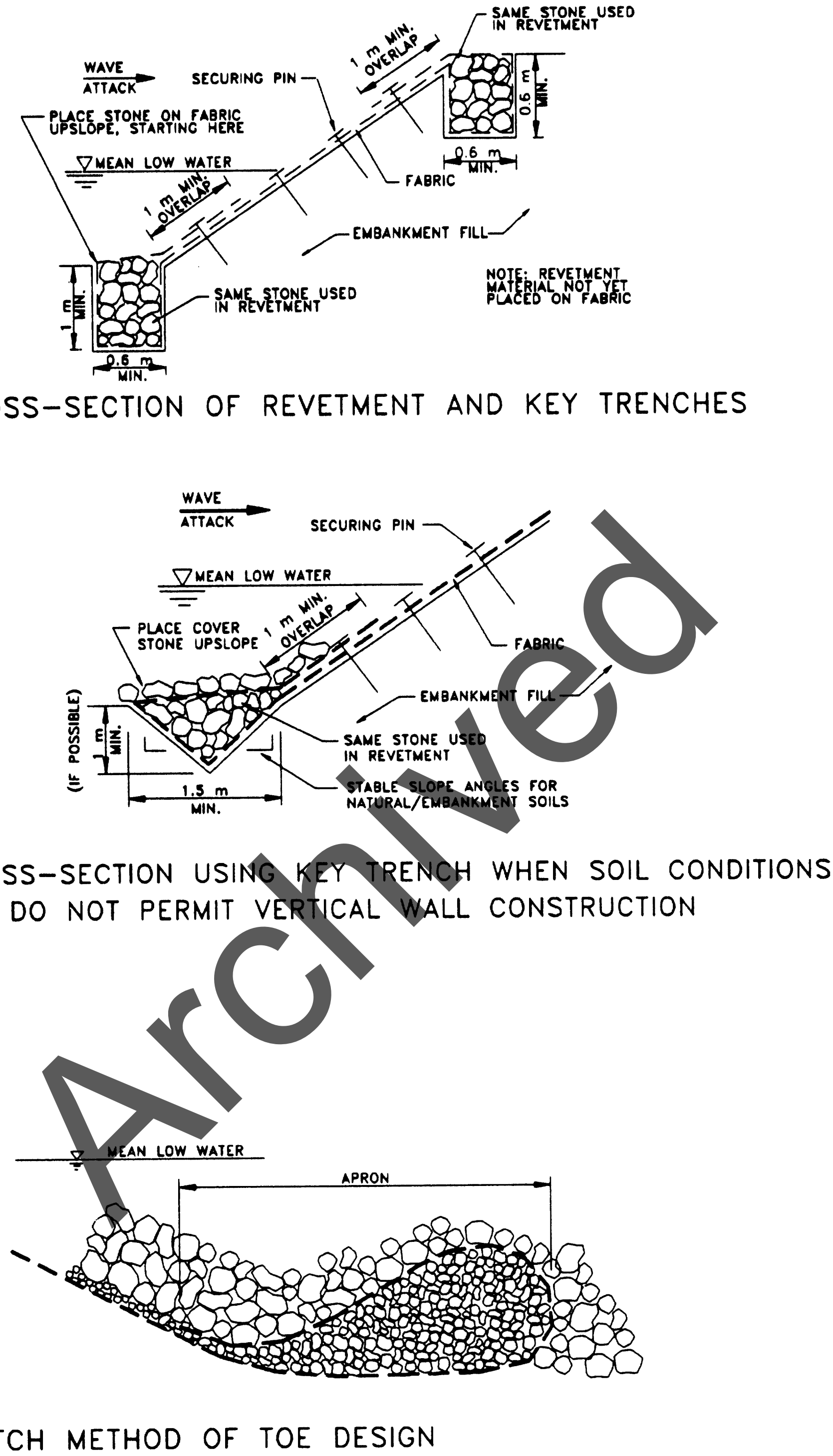


Figure 3-2 Construction of hard armor erosion control systems (a., b. after Keown and Dardeau, 1980; c. after Dunham and Barrett, 1974)

6. Place revetment (cushion layer and/or riprap) over the geotextile width, while avoiding puncturing or tearing it.
   a. Revetment should be placed on the geotextile within 14 days.
   b. Placement of armor cover will depend on the type of riprap, whether quarry stone, sandbags (which may be constructed of geotextiles), interlocked or articulating concrete blocks, soil-cement filled bags, or other suitable slope protection is used.
   c. For sloped surfaces, placement should always start from the base of the slope, moving up slope and, preferably, from the center outward.
   d. In no case should stone weighing more than 400 N be allowed to roll downslope on the geotextile.
   e. Field trials should be performed to determine if placement techniques will damage the geotextile and to determine the maximum height of safe drop. As a general guideline, for Moderate Survivability geotextiles (Table 3-1) with no cushion layer, height of drop for stones less than 100 kg should be less than 300 mm. For High Survivability geotextiles (Table 3-1) or Moderate Survivability geotextiles with a cushion layer, height of drop for stones less than 100 kg should be less than 0.9 m. Stones greater than 100 kg should be placed with no free fall unless field trials demonstrate they can be dropped without damaging the geotextile.
   f. Grading of slopes should be performed during placement of riprap. Grading should not be allowed after placement if it results in stone movement directly on the geotextile.

As previously indicated, construction requirements will depend on specific application and site conditions. In some cases, geotextile selection is affected by construction procedures. For example, if the system will be placed below water, a geotextile that facilitates such placement must be chosen. The geotextile may also affect the construction procedures. For example, the geotextile must be completely covered with riprap for protection from long-term exposure to ultraviolet radiation. Sufficient anchorage must also be provided by the riprap for *weighting* the geotextile in below-water applications. Other requirements related to specific applications are depicted in Figure 3-3 and are reviewed in the following subsections (from Christopher and Holtz, 1985).

3.8-2 Cut and Fill Slope Protection

Cut and fill slopes are generally protected using an armor stone over a geotextile-type system. Special consideration must be given to the steepness of the slope. After grading, clearing, and leveling a slope, the geotextile should be placed directly on the slope. When possible, geotextile placement should be placed parallel to the slope direction. A minimum overlap of 0.3 m between adjacent roll ends and a minimum 0.3 m overlap of adjacent strips is recommended. It is also important to place the up-slope geotextile over the down-slope geotextile to prevent overlap

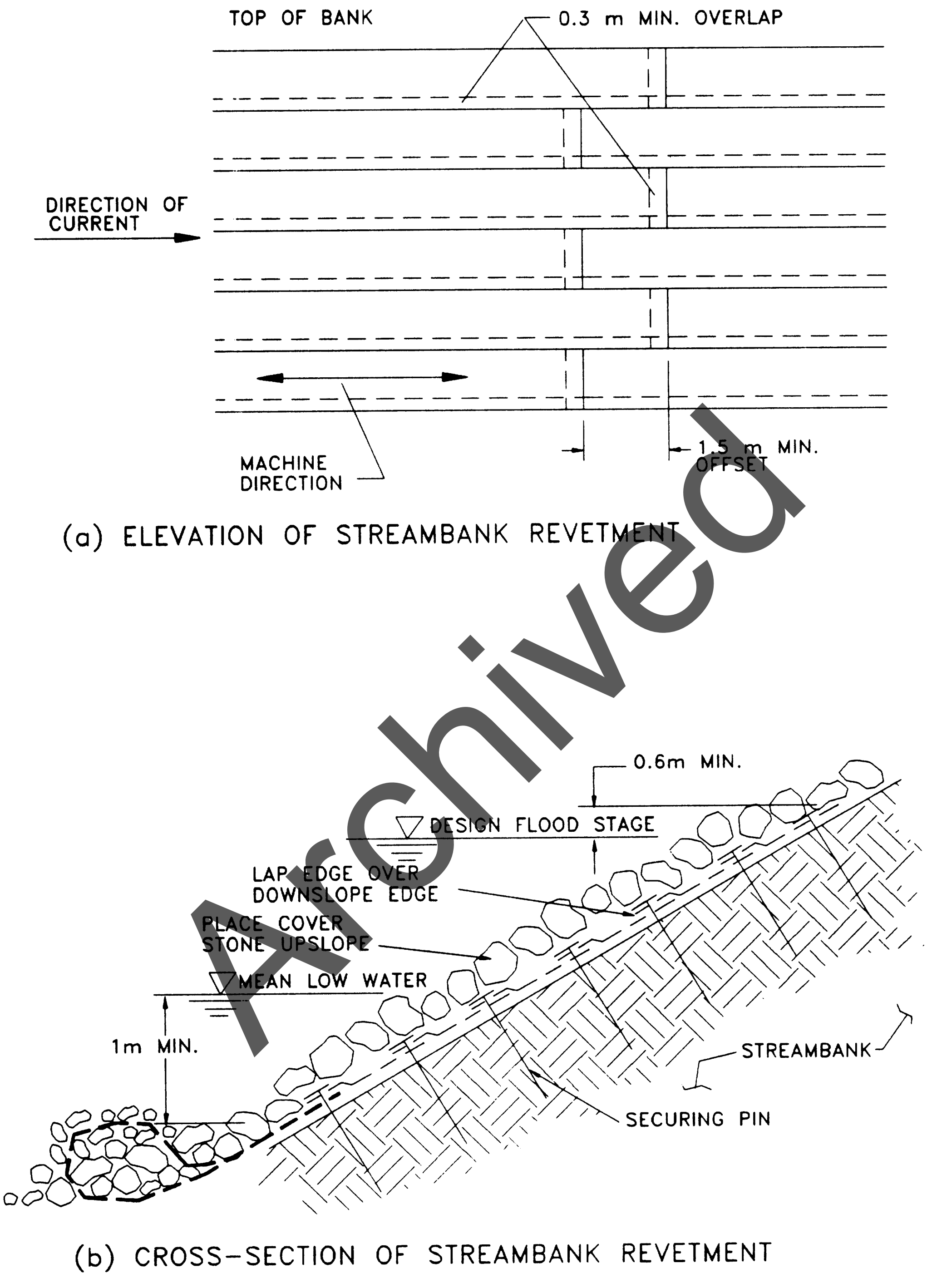
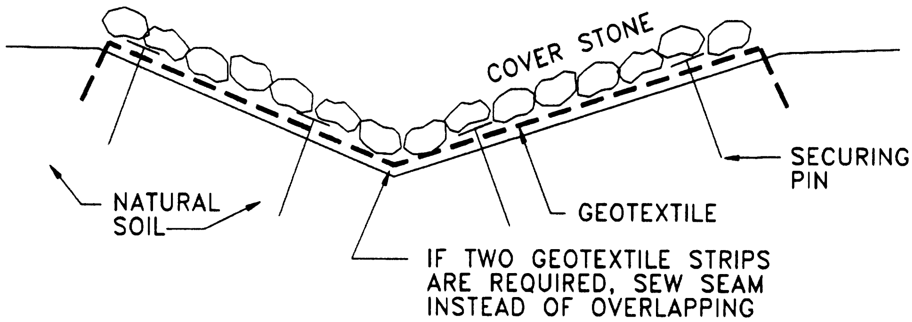


Figure 3-3 Special construction requirements related to specific hard armor erosion control applications.



(c) CROSS-SECTION OF GEOTEXTILE LINED DITCH

(d) SCOUR PROTECTION FOR ABUTMENTS

Figure 3-3 Special construction requirements related to specific hard armor erosion control applications (cont.).

separation during aggregate placement. When placing the aggregate, do not push the aggregate up the slope against the overlap. Generally, cut and fill slopes are protected with armor stone, and the recommended placement procedures in Section 3.8-1 should be followed.

3.8-3 Streambank Protection

For streambank protection, selecting a geotextile with appropriate clogging resistance to protect the natural soil and meet the expected hydraulic conditions is extremely important. Should clogging occur, excess hydrostatic pressures in the streambank could result in slope stability problems. Do not solve a surface erosion problem by causing a slope stability problem!

Detailed data on geotextile installation procedures and relevant case histories for streambank protection applications are given by Keown and Dardeau (1980). Construction procedures essentially follow the procedures listed in Section 3.8-1. The geotextile should be placed on the prepared streambank with the machine direction placed parallel to the bank (and parallel to the direction of stream flow). Adjacent rolls of geotextile should be seamed, sewed, or overlapped; if overlapped, secure the overlap with pins or staples. A 0.3 m overlap is recommended for adjacent roll edges, with the upstream roll edge placed over the downstream roll edge. Roll ends should be overlapped 1 m and offset as shown in Figure 3-3a. The upslope roll should overlap the downslope roll.

The geotextile should be placed along the bank to an elevation determined to be below mean low water level based on anticipated flow velocities in the stream. Existing agency design criteria for conventional nongeotextile streambank protection could be utilized to locate the toe of the erosion protection system. In the absence of other specifications, placement to a vertical distance of 1 m below mean water level, or to the bottom of the streambed for streams shallower than 1 m, is recommended. Geotextiles should either be placed to the top of the bank or at a given distance up the slope above expected high water level from the appropriate design storm event, including whatever requirements are normally used for conventional (nongeotextile) streambank protection systems. In the absence of other specifications, the geotextile should extend vertically a minimum of 0.5 m above the expected maximum water stage, or at least 1 m beyond the top of the embankment if less than 0.5 m above expected water level.

If strong water movements are expected, the geotextile must be toed in at the top and bottom of the embankment, or the riprap extended beyond the geotextile 0.5 m or more at the toe and the crest of the slope. If scour occurs at the toe and the rocks beyond the geotextile are undermined, they will in effect *toe into* the geotextile. The whole unit thus drops, until the toed-in section is stabilized. However, if the geotextile extends beyond the stone and scour occurs, the geotextile will flap in the water action, causing accelerated formation of a scour pit at the toe. Alternative toe treatments are shown in Figure 3-2. The trench methods in Figures 3-2a and 3-2b require excavating a trench at the toe of the slope. This may be a good alternative for new construction; however, it should be evaluated with respect to slope stability when a trench will be excavated at the toe of a potentially saturated slope below the water level. Keying in at the top can consist of burying the top bank edge of the geotextile in a shallow trench. This will provide resistance to undermining from infiltration of over-the-bank precipitation runoff, and also provide stability should a storm greater than anticipated occur. However, unless excessive quantities of runoff are expected and stream flows are relatively small, this step is usually omitted.

The armoring material (*e.g.*, riprap, sandbags, blocks) must be placed to avoid tearing or puncturing the geotextile, as indicated in Section 3.8-1.

3.8-4 Precipitation Runoff Collection and Diversion Ditches

Runoff drainage from cut slopes along the sides of roads and in the median of divided highways is normally controlled with one or more gravity flow ditches. Runoff from the pavement surface and shoulder slopes are collected and conveyed to drop inlets, stream channels, or other highway drainage structures. If a rock protection-geotextile system is used to control localized ditch erosion problems, select and specify the geotextile using the properties indicated in Table 3-1. Geotextile requirements for ditch linings are less critical than for other types of erosion protection, and minimum requirements for noncritical, nonsevere applications can generally be followed. If care is taken during construction, the protected strength requirements appear reasonable. The geotextile should be sized with AOS to prevent scour and piping erosion of the underlying natural soil and to be strong enough to survive stone placement.

The ditch alignment should be graded fairly smooth, with depressions and gullies filled and large stones and other debris moved from the ditch alignment. The geotextile should be placed with the machine direction parallel to the ditch alignment. Most geotextiles are available in widths of 2 m or more, and, thus, a single roll width of geotextile may provide satisfactory coverage on the entire ditch. If more than one roll width of geotextile is required, sew adjacent rolls together. This can be done by the manufacturer or on site. Again, for seams, the required strength of the seam should meet the minimum seam requirements in Table 3-1. The longitudinal seam produced by roll joining will run parallel with the ditch alignment. Geotextile widths should be ordered to avoid overlaps at the bottom of the ditch, since this is where maximum water velocity occurs. Roll ends should also be sewn or overlapped and pinned or stapled. If overlap is used, then an overlap of at least 1 m is recommended. The upslope roll end should be lapped over the downslope roll end, to retard in-service undermining. Pins or staples should be spaced so slippage will not occur during stone placement or after the ditch is placed in service.

Cover stone, sandbags, or other material intended to dissipate precipitation runoff energy should be placed directly on the geotextile, from downslope to upslope. Cover stone should have sufficient depth and gradation to protect the geotextile from ultraviolet radiation exposure. Again, the stone should be placed with care, especially if the geotextile strength criteria have been reduced to a *less critical* in-service application. A cross section of the proper placement is shown in Figure 3-3c. Vegetative cover can be established through the geotextile and stone cover if openings in the geotextile are sufficient to support growth. If a vegetative cover is desirable, geotextiles should be selected on the basis of the largest opening possible.

3.8-5 Wave Protection Revetments

Because of cyclic flow conditions, geotextiles used for wave protection systems should be selected on the basis of severe criteria, in most cases. Geotextile should be placed in accordance with the procedures listed in Section 3.8-1.

If a geotextile will be placed where existing riprap, rubble, or other materials placed on natural soil have been unsuccessful in retarding wave erosion, site preparation could consist of covering the existing riprap with a filter sand. The geotextile could then be designed with less rigorous requirements as a filter for the sand than if the geotextile is required to filter finer soils.

The geotextile is unrolled and loosely laid on the smooth graded slope. The machine direction of the geotextile should be placed parallel to the slope direction, rather than perpendicular to the slope, as was recommended in streambank protection. Thus, the long axis of the geotextile strips will be parallel to anticipated wave action. Sewing of adjacent rolls or overlapping rolls and roll ends should follow the steps described in Section 3.8-1, except that a 1 m overlap distance is recommended by the Corps of Engineers for underwater placement (Figure 3-2). Again, securing pins (requirements per Section 3.8-1) should be used to hold the geotextile in place.

If a large percentage of geotextile is to be placed below the existing tidal level, special fabrication and placement techniques may be required. It may be advantageous to pre-sew the geotextile into relatively large panels and pull the prefabricated panels downslope, anchoring them below the waterline. Depending upon the placement scheme used, selection of a floating or nonfloating geotextile may be advantageous.

Because of potential wave action undermining, the geotextile must be securely toed-in using one of the schemes shown in Figure 3-2. Also, a key trench should be placed at the top of the bank, as shown in Figure 3-2a, to prevent revetment stripping should the embankment be overtopped by wave action during high-level storm events.

Riprap or cover stone should be placed on the geotextile from downslope to upslope, and stone placement techniques should be designed to prevent puncturing or tearing of the geotextile. Drop heights should follow the recommendations stated in the general construction criteria (see 3.8-1).

3.8-6 Scour Protection

Scour, because of high stream flow around or adjacent to structures, generally requires scour protection for structures. Scour protection systems generally fall under the critical and/or severe design criteria for geotextile selection.

An extremely wide variety of transportation-associated structures are possible and, thus, numerous ways exist to protect such structures with riprap geotextile systems. A typical application is shown in Figure 3-3d. In all instances, the geotextile is placed on a smoothly graded surface as stated in the general construction requirements. Such site preparation may be difficult if the geotextile will be placed underwater, but normal stream action may provide a fairly smooth stream bed. In bridge pier protection or culvert approach and discharge channel protection applications, previous high-velocity stream flow may have scoured a depression around the structure. Depressions should be filled with granular cohesionless material. It is usually desirable to place the geotextile and rip rap in a shallow depression around bridge piers to prevent unnecessary constriction of the stream channel.

The geotextile should normally be placed with the machine direction parallel to the anticipated water flow direction. Seaming and/or overlapping of adjacent rolls should be performed as recommended in general construction requirements (Section 3.8-1). When roll ends are overlapped, the upstream ends should be placed over the downstream end. As necessary and appropriate, the geotextile may be secured in place with steel pins, as previously described. Securing the geotextile in the proper position may be of extreme importance in bridge pier scour protection. However, under high-flow velocities or under deep water, it will be difficult, if not impossible, to secure the geotextile with steel pins alone. Underwater securing methods must then be developed, and they will be unique for each project. Alternative methods include floating the geotextile into place, then filling from the center outward with stones, building a frame to which the geotextile can be sewn; using a heavy frame to submerge and anchor the geotextile; or constructing a light frame, then floating the geotextile and sinking it with riprap. In any case, it may be desirable to specify a geotextile which will either float or sink, depending upon the construction methods chosen. This can be based on a bulk density criteria for the geotextiles (*i.e.*, bulk density greater than 1 g/cm^3 will sink and less than 1 g/cm^3 will float).

Riprap and/or bedding material, precast concrete blocks, or other elements to be placed on the geotextile should be placed without puncturing or tearing the geotextile. Drop heights should be selected on the basis of geotextile strength criteria, as discussed in the general construction requirements (Section 8.3-1).

3.9 GEOTEXTILE FIELD INSPECTION

In addition to the general field inspection checklist presented in Table 1-4, the field inspector should pay close attention to construction procedures. If significant movement (greater than 0.15 m) of stone riprap occurs during or after placement, stone should be removed to inspect overlaps and ensure they are still intact. As indicated in Section 3.8, field trials should be performed to demonstrate that placement procedures will not damage the geotextile. If damage is observed, the engineer should be contacted, and the contractor should be required to change the placement procedure.

For below-water placement or placement adjacent to structures requiring special installation procedures, the inspector should discuss placement details with the engineer, and inspection requirements and procedures should be worked out in advance of construction.

3.10 GEOTEXTILE SELECTION CONSIDERATIONS

To enhance system performance, special consideration should be given to the type of geotextile chosen for certain soil and hydraulic conditions. The considerations listed in Section 2.10 also apply to erosion control systems. Special attention should be given to gap-graded soils, silts with sand seams, and dispersive clays. In certain situations, multiple filter layers may be appropriate. These consist of a sand layer over the soil, with the geotextile designed as a sand filter only and with sufficient size and number of openings to allow any fines that reach the geotextile to pass through it. Another special consideration for erosion control applications relates to preference toward felted versus slick geotextiles on steep slope sections. In any case, for steep slopes, the potential for riprap to slide on the geotextile must be assessed either through field trials or laboratory tests.

3.11 EROSION CONTROL MATS

In unlined areas where water can flow, the earth surface is susceptible to erosion by high-velocity flow. Where flow is intermittent, a grass cover will provide protection against erosion. By reinforcing the grass cover, the resulting composite armor layer will enhance the erosion resistance. Geosynthetic erosion control *mats* are made of synthetic meshes and webbings that reinforce the vegetation root mass to provide tractive resistance to high water velocities (*e.g.*, 6 m/s). *Mats* are used within this manual to describe geosynthetics for permanent erosion control applications, and *blankets* (see Chapter 4) are used to describe geosynthetics used in temporary applications (*i.e.*, until vegetation is established).

The three-dimensional erosion control mats retain soil, moisture, and seed, and thus promote vegetative growth. The principal applications of reinforced grass are in highway stormwater runoff ditches, steep waterways such as auxiliary spillways on dams, and protection of embankments against erosion by heavy precipitation or flooding events. Reinforced grass is used for temporary (*e.g.*, 2 hours), high-velocity flow areas, and not for permanent or long-term flow applications suited for hard armor systems. These systems have been found very effective in preventing erosion of the steep face of reinforced slopes (Chapter 8).

This section provides the general design and construction procedures and principles for grass systems reinforced with erosion control mats. The information contained in this section along with additional details pertaining to planning, design, specifications, construction, on-going management, and support research, are contained in Hewlett, Boorman and Bramley (1988).

The performance of reinforced grass is determined by a complex interaction of the constituent elements. At present, these physical processes, and the engineering properties of geotextiles and grass, cannot be fully described in quantitative terms. Thus, the design approach is largely empirical and involves a systematic consideration of each constituent element's behavior under service conditions, and how engineering properties can be effectively, yet safely, utilized. Specific products have been tested in laboratory flume tests to empirically quantify the tractive shear forces and velocities they can withstand as a function of flow time.

3.11-1 Planning

The planning stage involves assessing the feasibility of constructing a reinforced grass system in a particular situation and establishing the basic design parameters. The following points should be considered at this stage:

- overall concept of the waterway, and frequency and duration of flow;
- risk (acceptability of failure);
- design discharge and hydraulic loading;
- properties of subsoil;
- *dry* usage in normal no-flow conditions (*e.g.*, agricultural or amenity use, risk of vandalism);
- maintenance ability and requirements of the owner;
- appearance;
- capital and maintenance costs;
- access to site and method of construction;
- climate; and
- strategy for design, specification, construction, and future maintenance.

Any reinforced grass waterway will require an inspection and maintenance strategy different from that for conventionally lined waterways. Grass requires management, and some of the materials involved are more readily susceptible to damage, particularly by vandalism. If it is apparent at this stage that these considerations cannot be accommodated, then reinforced grass should not be used. However, the aesthetic advantages of a *soft* armor lining of reinforced grass usually outweighs potential disadvantages.

3.11-2 Design Procedure

Once the feasibility of constructing a reinforced grass waterway has been established, the detailed design can proceed. This will involve consideration of the hydraulic, geotechnical, and botanical aspects of the project. See by Hewlett, et. al. 1988, for other details.

Hydraulic Design: The main hydraulic design parameters are the velocity and duration of flow, as well as the erosion resistance of various armor layers.

The recommended hydraulic design procedure is as follows:

1. Choose the design hydrograph or overtopping condition. The consequences of waterway failure should be considered. Generally, grassed slopes can be considered where the overtopping discharge intensity is less than 0.005 $m^3/s/m$. Hardened protection should be used for greater discharge intensities.

2. Consider various engineering options for the proposed waterway, with particular reference to topography of the site. A site survey may be required if sufficient topographical information is not available. These options may relate to either general overtopping or construction of a purpose-made channel. Channel widths, slopes downstream of the crest, and, where appropriate, alternative weir lengths and crest levels may be considered.

3. If a reservoir is involved, carry out a flood routing calculation for each option. If a spillway is involved, check that the freeboard is adequate (including any allowance for waves). The operation frequency of the waterway should then be apparent. Modify the layout accordingly if occurrence of flow is more or less frequent than desired. The effect of waves and spray on areas adjacent to the waterway, along with the potential effect of the works on the area downstream, should be considered.

4. A variety of engineering options may be suitable at the site. The detailed hydraulics of each option should be investigated using the following procedure:

(i) Select an armor layer and a hydraulic roughness "n" value from Figure 3-4.

(ii) Solve Manning's equation by trial and error for design flow or discharge intensity, using different depths of flow to determine the velocity. (Manning's equation is commonly used in civil engineering applications to estimate the velocity and depth of flow in open channels.)

$$V = \frac{R^{2/3} S^{1/2}}{n}$$

where:

V = mean velocity of flow (m/s)

R = hydraulic radius (m) which equals cross-sectional area of flow divided by wetted perimeter

S = slope of the energy line

n = Manning's roughness coefficient (Figure 3-4)

Alternative forms of the equation for discharge and discharge intensity in a wide channel, respectively, are:

$$Q = \frac{A R^{2/3} S^{1/2}}{n}$$

$$q = \frac{d^{5/3} S^{1/2}}{n}$$

where:

Q = discharge (m^3/s)

A = area of flow (m^2)

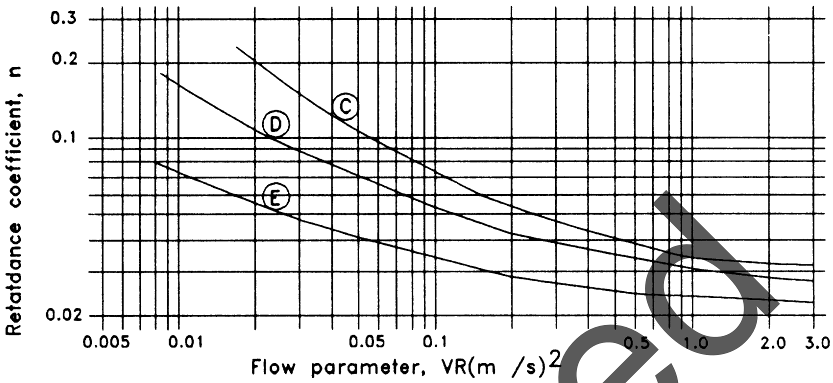
q = discharge per unit width of channel ($m^3/s/m$)

d = depth of flow (m)

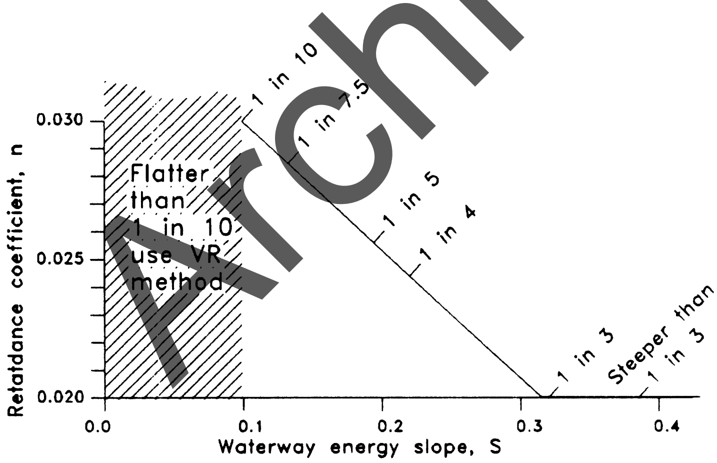
A channel may be considered to be *hydraulically* wide when velocity in the center of the channel is not affected by friction at the sides. In supercritical flow, this may require a channel width of up to 10 times the depth of flow.

Grass Retardance Catagories

Average grass length	Retardance
150 mm to 250mm	C
50mm to 150mm	D
less than 50mm	E



(a) HYDRAULIC ROUGHNESS OF GRASSES FOR SLOPES FLATTER THAN 1 IN 10



(b) RECOMMENDED RETARDANCE COEFFICIENTS FOR GRASSED SLOPES STEEPER THAN 1 ON 10

Figure 3-4 Roughness and retardance coefficients n for grassed slopes (Hewlett et al., 1987).

When uniform flow conditions have developed (*i.e.*, terminal velocity is reached), the energy slope, S_f equals the slope of the channel bed. Depth of uniform flow conditions is referred to as *normal depth*.

On steep slopes, the terminal velocity and normal *blackwater* depth calculated using Manning's equation will normally be achieved. The normal *blackwater* depth may be converted to *whitewater* using the air voids ratio. For water flow with a relatively small head loss between upstream and downstream energy levels, normal depth may not be reached; a step-by-step method should be used to determine the depth of flow and maximum velocity (Hewlett et al., 1987).

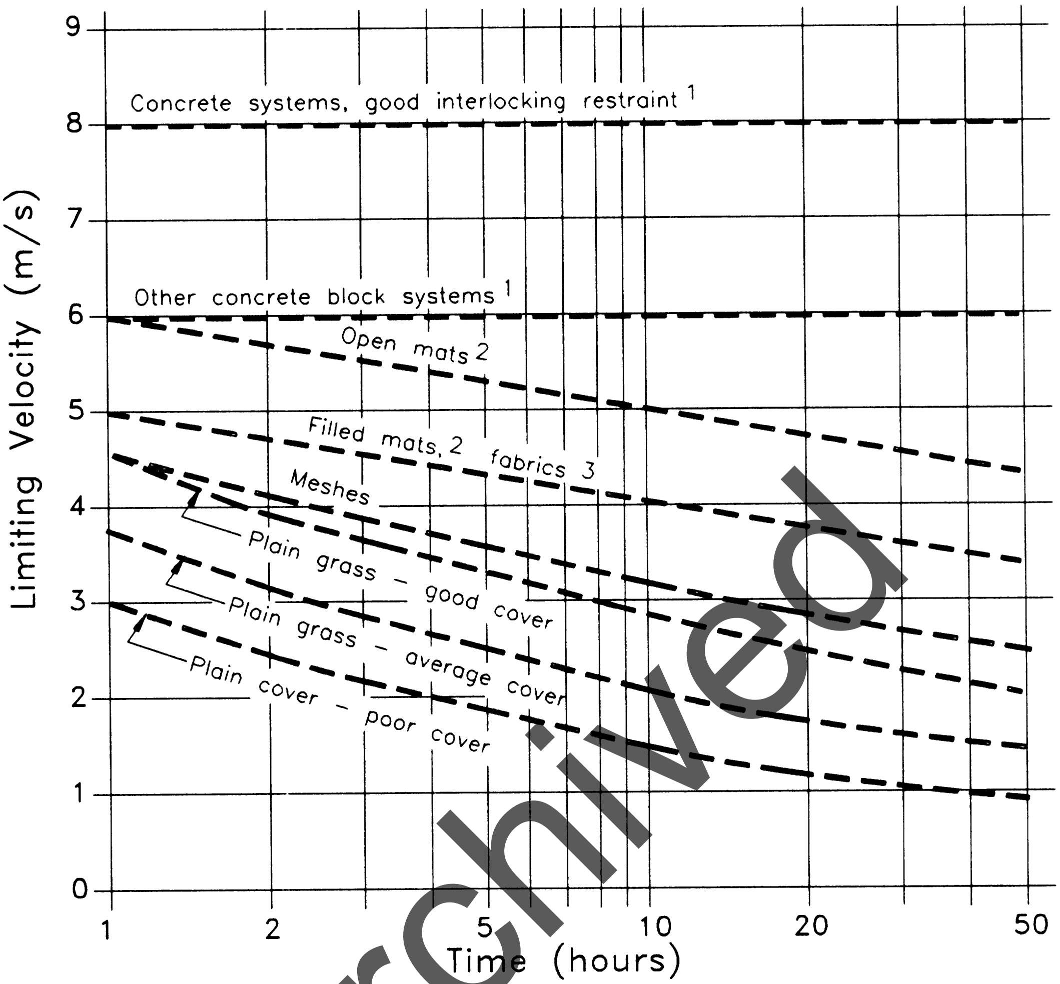
(iii) Compare this velocity with the recommended velocity for the armor layer from Figure 3-5. If the recommended velocity is exceeded, it may be possible to decrease the discharge intensity or select a more erosion-resistant armor layer. If the velocity is less than that recommended, it may be possible to reduce the base width or select a less erosion-resistant armor layer.

5. Determine the tailwater conditions over a range of discharges and consider ways to dissipate energy at the toe of the waterway.

If the tailwater conditions cause a hydraulic jump to form on the slope (Figure 3-6, Case (a)), it may be advisable to provide heavier armor, stronger restraint, discharge, or anchorage than normally used to protect the waterway from erosion by high-velocity flow. The decision will depend on the energy loss and frequency of occurrence. The critical zone of potential erosion is at the front of the jump. Experience from field trials and embankment overtopping under high tailwater conditions has shown that high-velocity flow zones within the jump generally occur only at the front of the jump and that erosion is consequently restricted.

If Cases (b), (c), or (d) in Figure 3-6 apply, provided the slope reinforcement is terminated in a safe manner, limited erosion may be acceptable. Note that in all cases, the flow velocity decreases downstream of the toe. Erosion protection may be provided -- either by continuing the slope reinforcement or by other means (*e.g.*, gabion mattress, rock armor).

If it is necessary to stabilize and contain the hydraulic jump -- for example, to accommodate the short-term design discharge -- then a control and/or armored stilling basin may be adopted.



Notes:
1. Minimum superficial mass 135 kg/m[2] .
2. Minimum nominal thickness 20 mm.
3. Installed within 20 mm of soil surface, or in conjunction with surface mesh.
4. See text for other criteria for geosynthetic reinforcement.
5. These graphs should only be used for erosion resistance to unidirectional flow. Values are based on available experiance and information as of 1987.
6. All reinforced grass values assume well established, good grass cover.
7. Other criteria (such as short term protection, ease of installation and management, susceptability to vandalism, etc) must be considered in choice of reinforcement.

EROSION RESISTANCE

Figure 3-5 Recommended limiting values for erosion resistance of plain and reinforced grass (Hewlett et al., 1987).

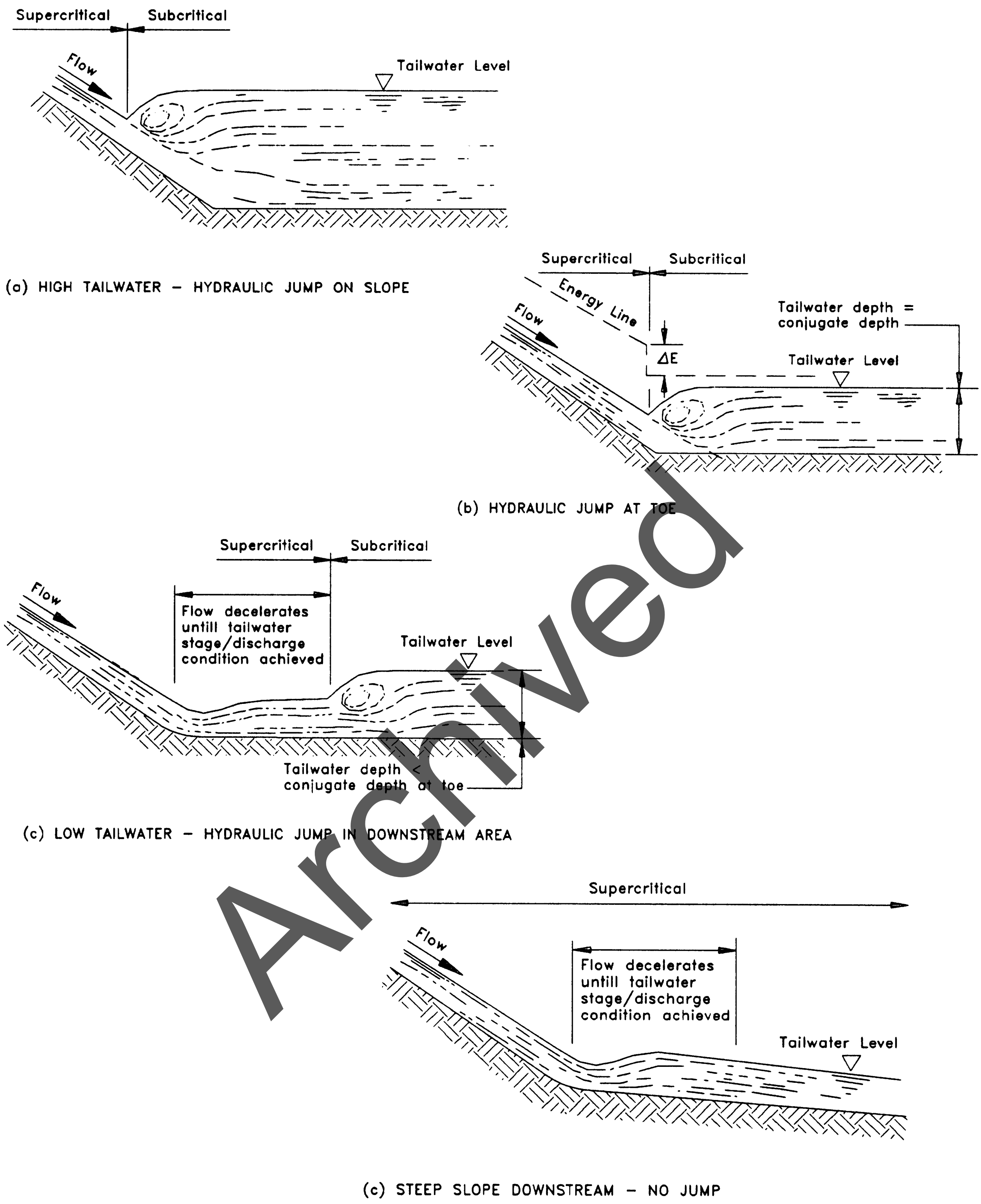


Figure 3-6 Possible flow conditions at base of steep waterway (Hewlett et al., 1987).

Geotechnical Considerations: The principal geotechnical consideration is the effect that water entering the embankment (or excavation) will have on the subsoil. The procedure normally followed is listed below. Consider the following principal points: (1) investigate the stability of the slope during normal *dry* conditions, as well as during and immediately following flow; (2) consider whether any localized drainage should be provided to provide relief of pore pressures for increased stability; and (3) consider whether there is likely to be any settlement of the subsoil and whether the armor layer is flexible enough to accommodate movement.

Botanical Considerations: Botanical considerations include the choice of grass mixture, and its establishment and management. Consider the following principal points.

1. Obtain samples of soil that will support the grass and carry out physical and chemical tests to determine its suitability.
2. Choose a grass mixture. The principal factors affecting this choice are soil conditions, climate, and management requirements.
3. Decide on the method of sowing and establishment of grass.

Detailing and Specification: A number of detailed points should be considered which combine the hydraulic, geotechnical, and botanical aspects, to complete the design process. These should be included on the drawings or in the specification and are listed below.

1. Anchorage: Anchorage details of geosynthetic erosion control mats should be developed, by the design engineer, on a project specific basis. Details include type and length of anchorage pins or stakes, spacing of anchors across and along the edges the mat, roll end anchorage, downslope shingling or anchorage of adjacent rolls, and anchorage at the top of slope or embankment.
2. Crest Details: Complete a detailed design of the waterway or slope crest. The upstream end of the reinforcement system must be designed to avoid the risk of waterway erosion from the upstream area.
3. Channel Details: Cross-sections of the channel should be drawn. Estimate freeboard based on bulked depth of flow. Careful detailing is required at any transition between two or more plane surfaces.
4. Toe Details: Complete a detailed design of the toe of the waterway or slope.
5. Construction Details: Foundation preparation, transition to adjacent structures, placement requirements, etc.

Details for each of these requirements are in Hewlett, et al. (1988). Remember to:

- check that the waterway will perform satisfactorily;
- produce the construction drawings;
- prepare a specification, including material and acceptance tests; and
- set up a framework for future construction, maintenance, and inspection.

It is important that adequate design and site supervision be exercised at all stages by the client or its representative to ensure that the work is constructed in accordance with good practice.

3.11-3 Specification

The following example specification for erosion control mats is after the Texas Department of Transportation specification for RECPs. This agency tests candidate erosion control materials and categorizes them into classes and types in an approved materials list.

SOIL EROSION CONTROL MATS

(after Texas Department of Transportation, Special Specification, Item 1225 February 1993)

1. DESCRIPTION.

This item shall govern for providing and placing wood, straw, or coconut fiber mat, synthetic mat, jute mesh or other material as a soil erosion control mat on slopes or ditches or for long-term protection of seeded areas as shown on the plans or as specified by the Engineer.

2. MATERIALS.

(1) Soil Erosion Control Mats. All soil erosion control mats must be prequalified by the Director of Maintenance and Operations prior to use.

Prequalification procedures and a current list of prequalified materials may be obtained by writing to the Director of Maintenance and Operations. A 0.3 m x 0.3 m sample of the material may be required by the Engineer in order to verify prequalification. Samples taken, accompanied by the manufacturer's literature, will be sent, properly wrapped and identified, to the Division of Maintenance and Operations for verification.

The soil erosion control mat shall be a Class 2 material and be one (1) of the following types as shown on the plans:

Class 2. "Erosion Control Mat"

(i) Type E. Short-term duration (Up to 2 Years)
Shear Stress (t_d) < 50 Pa

Prequalified Type E products are:

______ ______ ______
______ ______ ______
______ ______ ______

(ii) Type F. Short-term duration (Up to 2 Years)
Shear Stress (t_d) 50 to 95 Pa

Prequalified Type F products are:

(iii) Type G. Long-term duration (Longer than 2 Years)
Shear Stress (t_d) > 95 to < 240 Pa

Prequalified Type G products are:

(iv) Type H. Long-term duration (Longer than 2 Years)
Shear Stress (t_d) greater than or equal to 240 Pa

Prequalified Type H products are:

(2) Staples. Staples for anchoring the soil erosion control mat shall be U-shaped, made of 3 mm or large diameter steel wire, or other approved material, have a width of 25 to 50 mm, and a length of not less than 150 mm for firm soils and not less than 300 mm for loose soils. [Longer staples, and closer spacings, should be considered for steep reinforced soil slope applications.]

3. CONSTRUCTION METHODS.

(1) General. The soil erosion control mat shall conform to the class and type shown on the plans. The Contractor has the option of selecting an approved soil erosion control mat conforming to the class and type shown on the plans, and according to the current approved material list.

(2) Installation. The soil erosion control mat, whether installed as slope protection or as flexible channel liner in accordance with the approved materials list, shall be placed within 24 hours after seeding or sodding operations have been completed, or as approved by the Engineer. Prior to placing the mat, the area to be covered shall be relatively free of all rocks or clods over 1-½ inches in maximum dimension and all sticks or other foreign material which will prevent the close contact of the mat with the soil. The area shall be smooth and free of ruts or depressions exist for any reason, the Contractor shall be required to rework the soil until it is smooth and to reseed or resod the area at the Contractor's expense.

Installation and anchorage of the soil erosion control mat shall be in accordance with the project construction drawings unless otherwise specified in the contract or directed by the Engineer.

(3) Literature. The Contractor shall submit one (1) full set of manufacturer's literature and manufacturer's installation recommendations for the soil erosion control mat selected in accordance with the approved material list.

4. MEASUREMENT.

This Item will be measured by the square meter of surface area covered.

5. PAYMENT.

The work performed and materials furnished in accordance with this Item and measured as provided under "Measurement" will be paid for at the unit price bid for "Soil Erosion Control Mat" of the class and type shown on the plans. This price shall be full compensation for furnishing all materials, labor, tools, equipment and incidentals necessary to complete the work. Anchors, checks, terminals or junction slots, and wire staples or wood stakes will not be paid for directly but will be considered subsidiary to this Item.

3.12 REFERENCES

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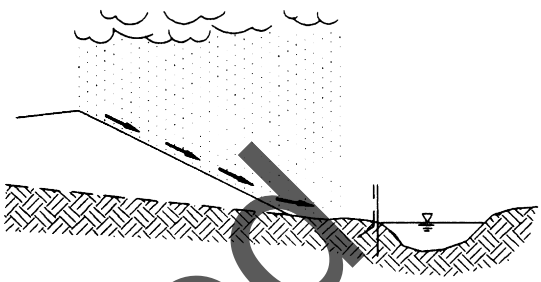
4.0 TEMPORARY RUNOFF AND SEDIMENT CONTROL

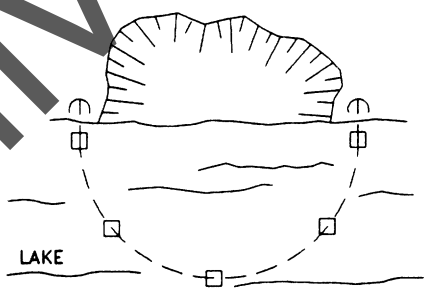
4.1 INTRODUCTION

Geotextiles, geosynthetic erosion control blankets, and other geosynthetic products can be used to temporarily control and minimize erosion and sediment transport during construction. Four specific application areas have been identified:

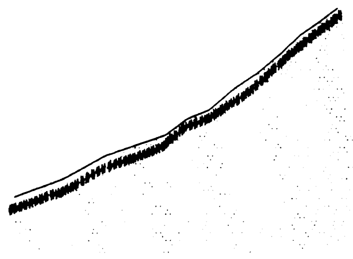
- Geotextile silt fences can be used as a substitute for hay bales or brush piles to remove suspended particles from sediment-laden runoff water.



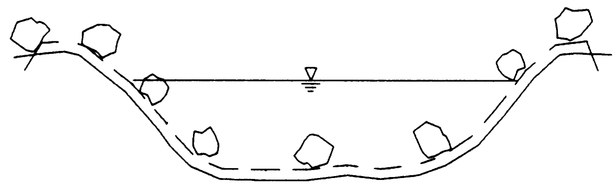
- Geotextiles can be used as a turbidity curtain placed within a stream, lake, or other body of water to retain suspended particles and allow sedimentation to occur.



- Special soil retention *blankets*, made of both natural and synthetic grids, meshes, nets, fibers, and webbings, can be used to provide tractive resistance and resist water velocity on slopes. These products retain seeds and add a mulch effect to promote the establishment of a vegetative cover.



- Geotextiles held in place by pins or riprap can be used to temporarily control erosion in diversion ditches, culvert outfalls, embankment slopes, etc. Alternatively, soil retention *blankets* can be used for temporary erosion control until vegetation can be established in the ditch.



The main advantages of using geosynthetics over conventional techniques in sediment control applications include the following.

- In the case of a silt fence, the geotextile can be designed for the specific application, while conventional techniques are basically designed by trial-and-error.
- Geotextile silt fences in particular often prove to be very cost-effective, especially in comparison to hay bales, considering ease of installation and material costs.
- Control by material specifications is easier.

For runoff control, geosynthetic products are designed to help mitigate immediate erosion problems and provide long-term stabilization by promoting the establishment and sustainment of vegetative cover. The main advantages of using geosynthetics for erosion control applications include the following.

- Vegetative systems have desirable aesthetics.
- Products are lightweight and easy to handle.
- Temporary, degradable products improve establishment of vegetation.
- Continuity of protection is generally better over the entire protected area.
- Empirically predictable performance; traditional techniques such as seeding, mulch covers, and brush or hay bale barriers, are often less reliable.

The following sections review the function, selection specifications, and installation procedures for geosynthetics used as silt fences, turbidity curtains, and erosion control blankets. Design of geotextiles in temporary riprap-geotextile systems to control ditch erosion follows Chapter 3 design guidelines. Additional information on erosion and sediment control will be available in the FHWA course and text entitled Identifying and Controlling Sedimentation and Erosion currently being developed.

4.2 FUNCTION OF SILT FENCES

In most applications, a geotextile silt fence is placed downslope from a construction site or newly graded area to reduce sediment being transported by runoff to the surrounding environment. Sometimes silt fences are used in permanent or temporary diversion ditches for the same purpose.

A silt fence primarily functions as a temporary dam (Mallard and Bell, 1981). It retains water long enough for suspended fine sand and coarse silt particles in the runoff to settle out before they reach the fence. Generally, a retention time of 20 to 25 minutes is sufficient, so flow through the geotextile after the first charge must provide this retention time. Although smaller geotextile pore opening sizes and low permittivity can be selected to allow finer particles to settle out, some water must be able to pass through the fence to prevent possible overtopping of the fence. A silt fence is intended for drainage areas experiencing sheet flow. Appropriate applications of silt fences are: along the site perimeter; below disturbed areas subject to sheet and rill erosion and sheet flow; and below the toe of exposed and erodible slopes.

Because not all the silt and clay in suspension will settle out before reaching the fence, water flowing through the fence will still contain some fines in suspension. Removal of fines by the geotextile creates a difficult filtration condition. If the openings in the geotextile (*i.e.*, AOS) are small enough to retain most of the suspended fines, the geotextile will blind and its permeability will be reduced so that bursting or overtopping of the fence could occur. Therefore, it is better to have some geotextile openings large enough to allow silt-sized particles to easily pass through. Even if some silt passes through the fence, the flow velocity will be small, and some fines may settle out. If the application is critical, *e.g.*, when the site is immediately adjacent to environmentally sensitive wetlands or streams, multiple silt fences could be used. A second fence with a smaller AOS is placed a short distance downslope of the first fence to capture silt that passed through the first fence.

In the past, the AOS and permittivity, ψ, have been used to design and specify the filtration requirements of the geotextile. However, Wyant (1980) and Allen (1994) indicate that these geotextile index properties are not directly related to silt fence performance. Experience indicates that, in general, most geotextiles have hydraulic characteristics that provide acceptable silt fence performance for even the most erodible silts (Wyant, 1980; Allen, 1994). Thus, geotextile selection and specification can be based on typical properties of silt fence geotextiles known to have performed satisfactorily in the past, or through the use of performance type tests such as ASTM D 5141, Determining Filtering Efficiency and Flow Rate of a Geotextile for Silt Fence Applications Using Site-Specific Soil. Past experience is the basis for the AOS and permittivity values presented later in this chapter.

Most silt fence applications are temporary; the fence only must work until the site can be revegetated or otherwise protected from rainfall and erosion. According to Richardson and Middlebrooks (1991), silt fences are best limited to applications where sheet erosion occurs and where flow is not concentrated, though silt fences can be used in both ditch or swale applications by special design (with varying success). Flow velocity should be less than about 0.3 m/s. Recommendations for allowable slope length versus slope angle to limit runoff velocity are presented in Table 4-1. Furthermore, the limiting slope angle and velocity requirements suggest that the drainage areas for overland flow to a fence should be less than about 1 ha per 30 m of fence.

Silt fence ends should be turned uphill to ensure they capture runoff water and prevent flow around the ends. The groundline at the fence ends should be at or above the elevation of the lowest portion of the fence top. Measures should be taken to prevent erosion along the fence backs that run downhill for a significant distance. Gravel check dams at approximately 2 to 3 m intervals along the back of the fence can be used.

TABLE 4-1
LIMITS OF SLOPE STEEPNESS AND LENGTH
TO LIMIT RUNOFF VELOCITY TO 0.3 m/s
(after Richardson and Middlebrooks, 1991)

Slope Steepness (%)	Maximum Slope Length (m)
< 2	30
2 - 5	25
5 - 10	15
10 - 20	10
> 20	5

4.3 DESIGN OF SILT FENCES

4.3-1 Simplified Design Method

This section follows the simplified design method of Richardson and Middlebrooks (1991), except the Revised Universal Soil Loss Equation (RUSLE) is used in Step 2 in lieu of the Universal Soil Loss Equation (USLE). See their paper for additional details on this design procedure. See the FHWA *Identifying and Controlling Erosion and Sedimentation* course text (Hydrodynamics, 1997) for a summary discussion on advantages and disadvantages of USLE and RUSLE equations.

STEP 1. Estimate runoff volume.

Use the Rational Method (small watershed areas):

$$Q = 2.8 \times 10^{-3}\ C\ i\ A \qquad [4 - 1]$$

where:

Q	=	runoff (m^3/s)
C	=	surface runoff coefficient
i	=	rainfall intensity (mm/hr)
A	=	area (ha)

Use C = 0.2 for rough surfaces, and C = 0.6 for smooth surfaces. A 10-year storm event is typically used for designing silt fences.

Use the appropriate rainfall intensity factor, i, for the locality. Assume a 10-year design storm, or use local design regulations. Neglect any concentration times (worst case). This calculation gives the total storage volume required of the silt fence.

STEP 2. Estimate sediment volume.

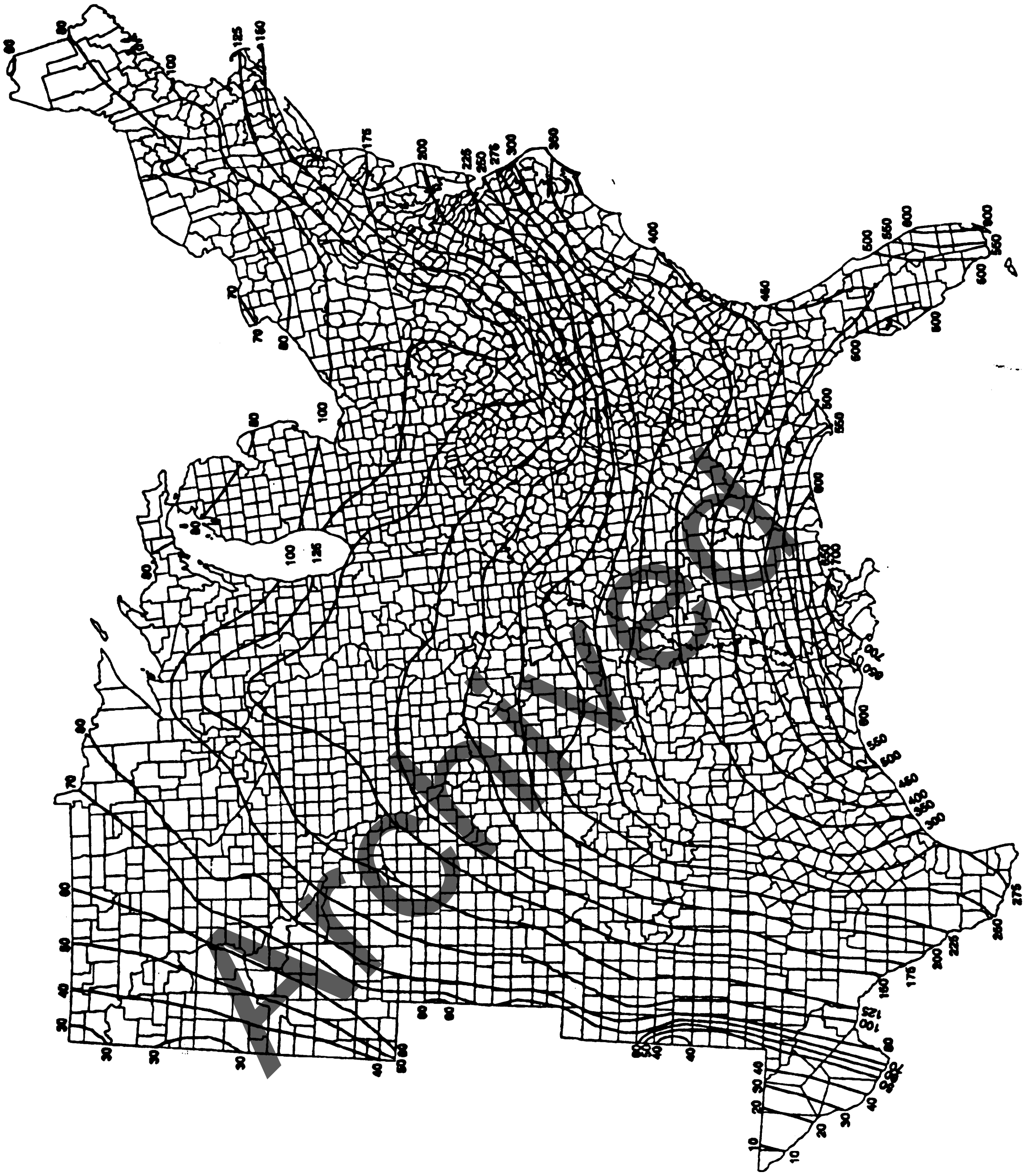
Use the Revised Universal Soil Loss Equation (RUSLE)

$$A = 2.2\ R\ K\ (LS)\ C\ P \qquad [4 - 2]$$

where:

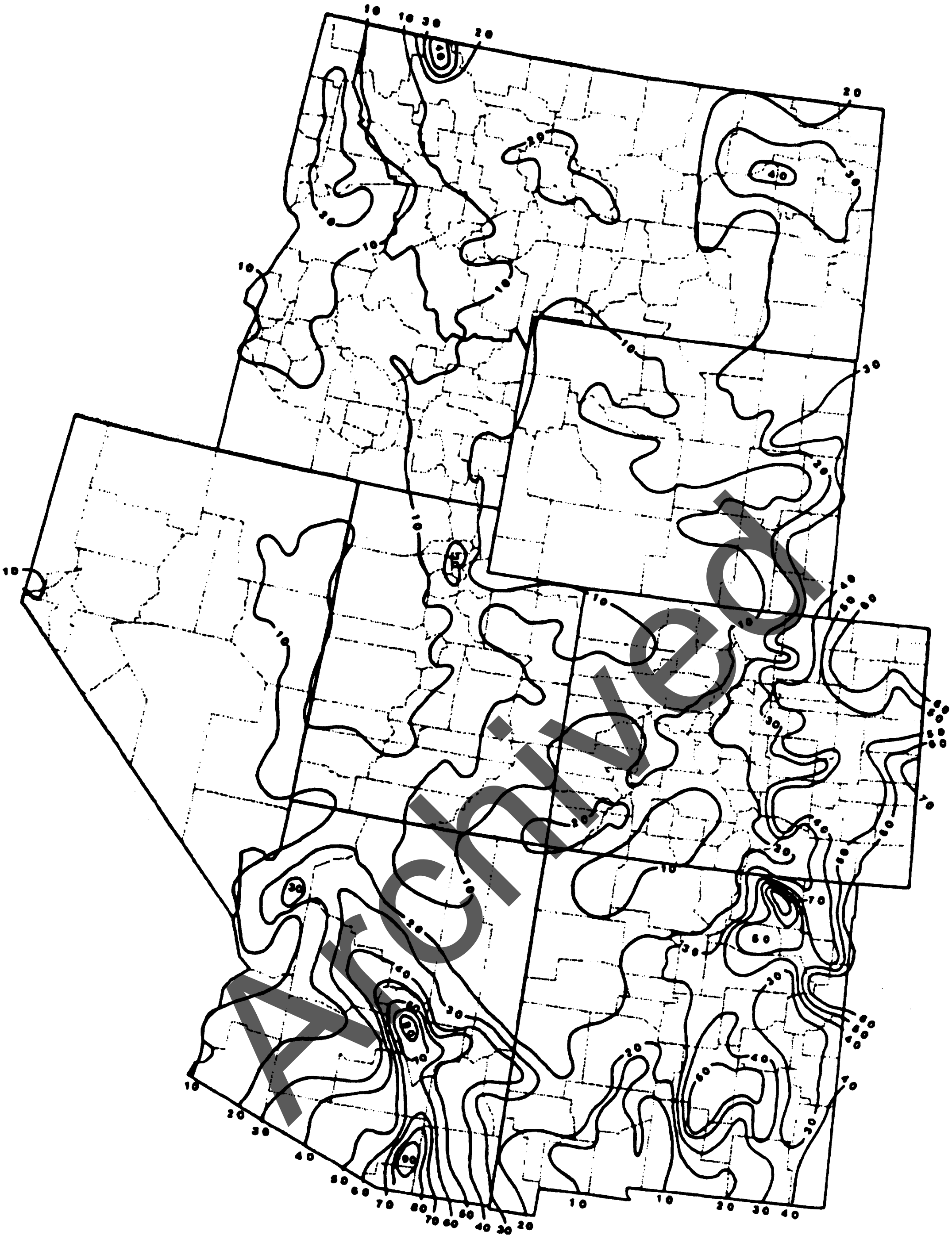
A	=	annual soil loss due to erosion (metric tons/ha/yr)
R	=	rainfall factor
K	=	soil erodibility factor
LS	=	slope length and steepness factor
C	=	vegetative cover factor © = 1 for no cover)
P	=	erosion control practice factor (P = 1 for minimal practice)

Obtain rainfall erosion index from Figure 4-1; note that the factors are based upon a 2-year, 6-hour storm event. Use Figure 4-2 to obtain the values of KLS (limited slope lengths and steepness factors are applicable to most silt fence applications).



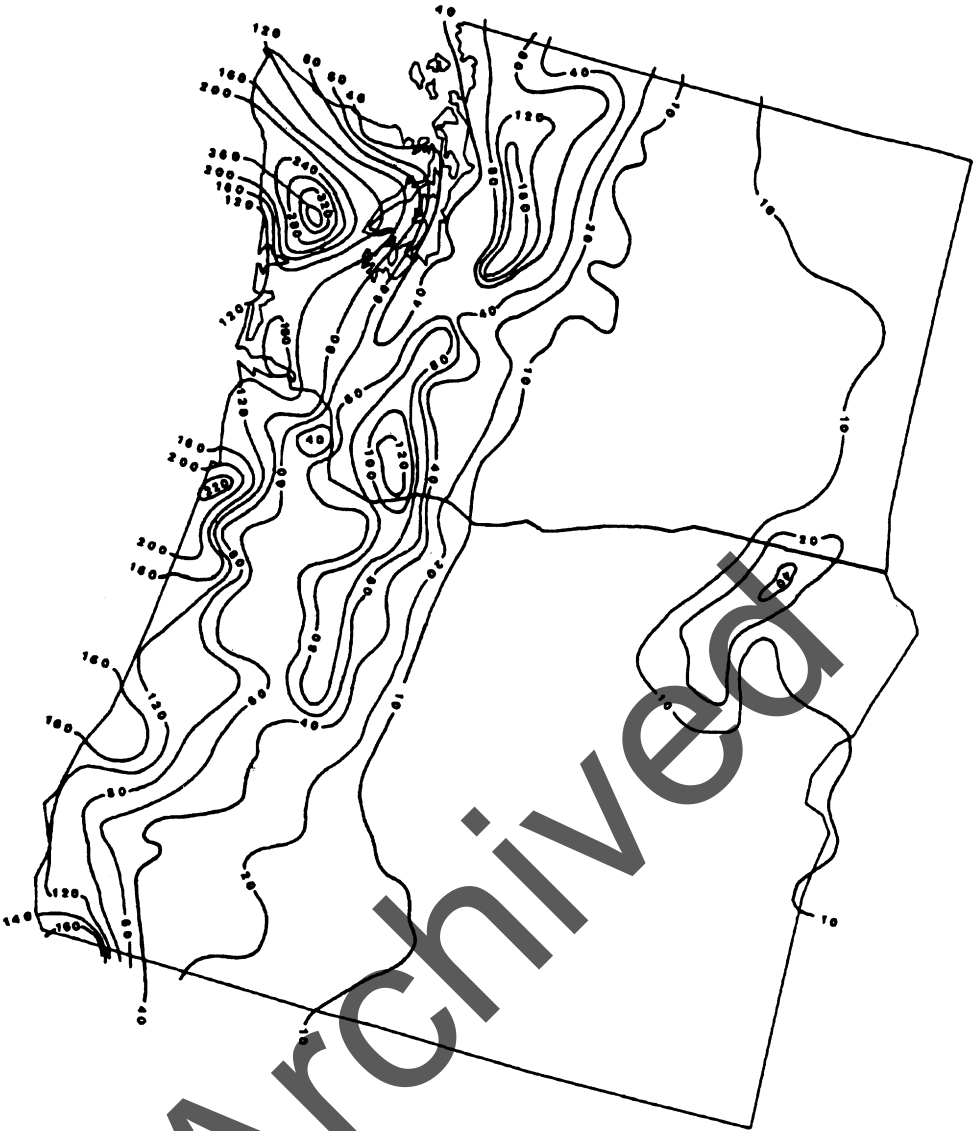
(a) annual R-factors for the eastern U.S.

Figure 4-1 Rainfall erosion factors, R (Renard et al., 1997).



(b) annual R-factors for the western U.S.

Figure 4-1 Rainfall erosion factors, R (Renard et al., 1997) (cont.).



(c) annual R-factors for Oregon and Washington

Figure 4-1 Rainfall erosion factors, R (Renard et al., 1997) (cont.).

(d) annual R-factors for California

Figure 4-1 Rainfall erosion factors, R (Renard et al., 1997) (cont.).

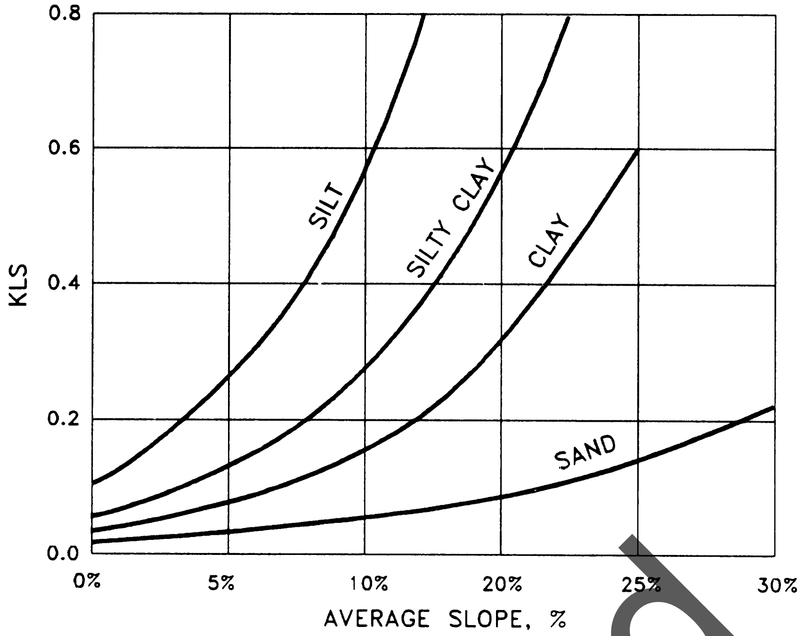


Figure 4-2 Universal soil loss KLS vs slope (Richardson and Middlebrooks, 1991).

Equation 4-2 predicts an erosion rate per year. This rate may be used to provide an estimate of predicted tons of sediment produced per hectare for a 6-month (typical) silt fence design (Richardson and Middlebrooks, 1991). This should provide a reasonable estimate for sizing the storage volume behind the silt fence. A density of about 800 kg/m^3 may be assumed for converting the soil loss in metric tons to a volume. Sediment behind a silt fence should be removed when accumulation reaches approximately one-third to one-half fence height.

STEP 3. Select geotextile.

A. Hydraulic properties

Because site specific designs for retention and permittivity are not necessary for most soils (at least in a practical sense), use nominal AOS and permittivity values for geotextiles known to perform satisfactorily as silt fences. Suggested values (Richardson and Middlebrooks, 1991) are:

0.15 mm < AOS < 0.60 mm for woven silt films

0.15 mm < AOS < 0.30 mm for all other geotextiles

Permittivity, ψ > 0.02 s^{-1}

B. Physical and mechanical properties

The geotextile must be strong enough to support the pooled water and the sediments collected behind the fence. Minimum strength depends on height of impoundment and spacing between fence posts.

Use Figure 4-3 to determine required tensile strength for a range of impoundment heights and post spacings. For geotextiles without wire or plastic mesh backing, limit impoundment heights to 0.6 m and post spacing to 2 m; for greater heights and spacings, use steel or plastic grid/mesh reinforcement to prevent burst failure of geotextile. Unsupported geotextiles must not collapse or deform, allowing silt-laden water to overtop the fence. Use Figure 4-4 to design the fence posts.

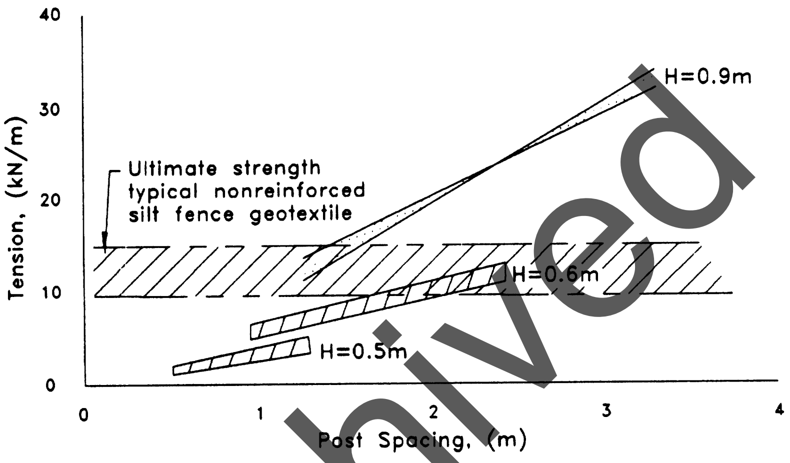


Figure 4-3 Geotextile strength versus post spacing (Richardson and Koerner, 1990).

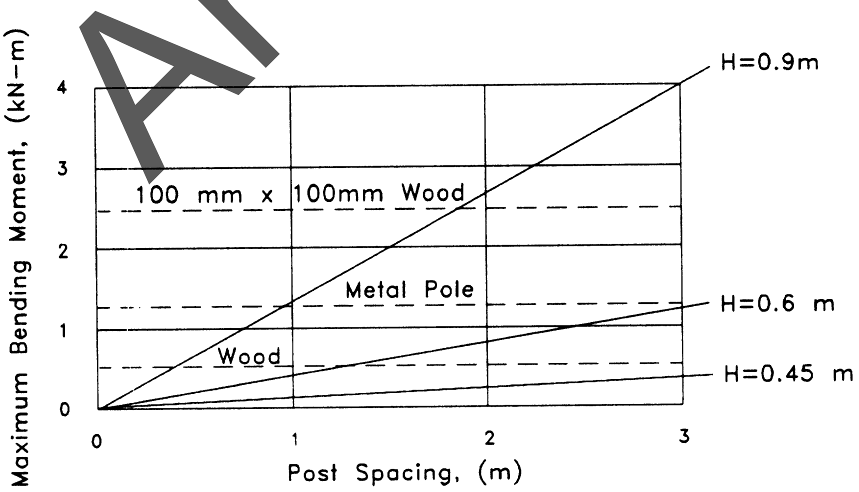


Figure 4-4 Post requirements vs post spacing (Richardson and Koerner, 1990).

4.3-2 Alternate Hydraulic Design Using Performance Tests

An alternate design approach for silt fences uses model studies to estimate filtration efficiency for specific site conditions. This method was developed by Wyant (1980) for the Virginia Highway and Transportation Research Council (VHTRC) and is based on observed field performance and laboratory testing. The procedures for this method are described in ASTM D 5141. The laboratory model consists of a flume with an outflow opening similar to the size of a hay bale and positioned at a fixed slope of 8%. The geotextile is strapped across the end of the flume. A representative soil sample from the site is then suspended in water to a concentration of about 3000 ppm (equivalent water content is 0.3 percent) and poured through the flume. Based on the performance of the geotextile, appropriate geotextiles can be selected to provide filtering efficiencies approximating of 75% or more and flow rates on the order of 0.1 L/min/m^2 after three test repetitions.

The model study approach provides a system performance evaluation by utilizing actual soils from the local area of interest. Thus, it cannot be performed by manufacturers. The approach lends itself to an *approved list*-type specification for silt fences. In this case, the agency or its representatives perform the flume test using their particular problem soils and prequalifies the geotextiles that meet filtering efficiency and flow criteria requirements. Qualifying geotextiles can be placed on an approved list that is then provided to contractors. Geotextiles on any approved list should be periodically retested because manufacturing changes often occur.

4.3-3 Constructability Requirements

The geotextile used as a silt fence must be strong enough to enable it to be properly installed. AASHTO M288 property recommendations are indicated in Table 4-2. Realize that these specifications are not based on research but on properties of existing geotextiles which have performed satisfactorily in silt fence applications. Also given are requirements for resistance to ultraviolet degradation. Although the applications are temporary (*e.g.*, 6 to 36 months), the geotextile must have sufficient UV resistance to function throughout its anticipated design life.

TABLE 4-2
PHYSICAL REQUIREMENTS[1,2,3]
FOR TEMPORARY SILT FENCE GEOTEXTILES
(AASHTO, 1997)

	ASTM Test Method	Units	Requirement		
			Supported[4] Silt Fence	Unsupported Silt Fence	
				Geotextile Elongation ≥ 50%[5]	Geotextile Elongation < 50%[5]
Maximum Post Spacing			1.2 m	1.2 m	1.2 m
Grab Strength Machine Direction X-Machine Direction	D 4632	N	 400 400	 550 450	 550 450
Permittivity[6]	D 4491	sec^{-1}	0.05	0.05	0.05
Apparent Opening Size	D 4751	mm	0.60 max.	0.60 max.	0.60 max.
Ultraviolet Stability (Retained Strength)	D 4355	%	70% after 500 hours of exposure	70% after 500 hours of exposure	

NOTES:
1. Acceptance of geotextile material shall be based on ASTM D 4759.
2. Acceptance shall be based upon testing of either conformance samples obtained using Procedure A of ASTM D 4354, or based on manufacturer's certifications and testing of quality assurance samples obtained using Procedure B of ASTM D 4354.
3. All numeric values except AOS represent minimum average roll value (*i.e.*, test results from any sampled roll in a lot shall meet or exceed the minimum values in the table). Lot samples according to ASTM D 4354.
4. Silt fence support shall consist of 14 gage steel wire mesh spacing of 150 mm by 150 mm or prefabricated polymeric mesh of equivalent strength.
5. As measured in accordance with ASTM D 4632.
6. These default filtration property values are based on empirical evidence with a variety of sediments. For environmentally sensitive areas, a review of previous experience and/or site or regionally specific geotextile tests should be performed by the agency to confirm suitability of these requirements.

4.4 SPECIFICATIONS

The following specifications were developed by the Washington State Department of Transportation in 1994 and are included herein for your reference. They are meant to serve as guidelines for selecting and installing of geotextiles for routine (less critical) projects. They are not intended to replace site-specific evaluation, testing, and design.

WASHINGTON STATE DEPARTMENT OF TRANSPORTATION
MATERIALS LABORATORY
TUMWATER, WA
GEOTEXTILE FOR SILT FENCE
1994

Description

The Contractor shall furnish and place construction geotextile for silt fence in accordance with the details shown in the Plans.

Materials

Geotextile and Thread for Sewing

The material shall be a geotextile consisting only of long chain polymeric fibers or yarns formed into a stable network such that the fibers or yarns retain their position relative to each other during handling, placement, and design service life. At least 85 percent by weight of the material shall be polyolefins or polyesters. The material shall be free from defects or tears. The geotextile shall also be free of any treatment or coating which might adversely alter its hydraulic or physical properties after installation. The geotextile shall conform to the properties as indicated in Table 1.

Thread used for sewing shall consist of high strength polypropylene, polyester, or polyamide. Nylon threads will not be allowed. The thread used to sew permanent erosion control geotextiles must also be resistant to ultraviolet radiation.

Table 1: Geotextile Property Requirements[1]
for Temporary Silt Fence

Geotextile Property	ASTM Test Method[2]	Unsupported Between Posts	Supported Between Posts with Wire or Polymeric Mesh
AOS	D 4751	0.15 mm min.; 0.30 mm max. for other geotextiles; 0.60 mm max. for slit film wovens	0.15 mm min.; 0.30 mm max. for other geotextiles; 0.60 mm max. for slit film wovens
Water Permittivity	D 4491	0.02 sec^{-1} min.	0.02 sec^{-1} min.
Grab Tensile Strength, min. in MD and CMD	D 4632	800 N min. in MD 450 N min. in CMD	450 N min.
Grab Failure Strain, min. in MD only	D 4632	30% max. at 800 N or more	---
Ultraviolet (UV) Radiation Stability	D 4355	70% Strength Retained min., after 500 hr in weatherometer	70% Strength Retained min., after 500 hr in weatherometer

NOTES:
1. All geotextile properties in Table 1 are minimum average roll values (*i.e.*, the test result for any sampled roll in a lot shall meet or exceed the values shown in the table).
2. The test procedures used are essentially in conformance with the most recently approved ASTM geotextile test procedures, except for geotextile sampling and specimen conditioning, which are in accordance with WSDOT Test Methods 914 and 915, respectively. Copies of these test methods are available at the Headquarters Materials Laboratory in Tumwater.

Geotextile Approval and Acceptance

Source Approval

The Contractor shall submit to the Engineer the following information regarding each geotextile proposed for use:

Manufacturer's name and current address,
Full product name,
Geotextile structure, including fiber/yarn type, and
Proposed geotextile use(s).

If the geotextile source has not been previously evaluated, a sample of each proposed geotextile shall be submitted to the Headquarters Materials Laboratory in Tumwater for evaluation. After the sample and required information for each geotextile type have arrived at the Headquarters Materials Laboratory in Tumwater, a maximum of 14 calendar days will be required for this testing. Source approval will be based on conformance to the applicable values from Tables 1 through 6. Source approval shall not be the basis of acceptance of specific lots of material unless the lot sampled can be clearly identified and the number of samples tested and approved meet the requirements of WSDOT Test Method 914.

Geotextile Samples for Source Approval

Each sample shall have minimum dimensions of 1.5 meters by the full roll width of the geotextile. A minimum of 6 square meters of geotextile shall be submitted to the Engineer for testing. The geotextile machine direction shall be marked clearly on each sample submitted for testing. The machine direction is defined as the direction perpendicular to the axis of the geotextile roll. Source approval for temporary silt fences will be by manufacturer's certificate of compliance as described under "Acceptance Samples."

The geotextile samples shall be cut from the geotextile roll with scissors, sharp knife, or other suitable method which produces a smooth geotextile edge and does not cause geotextile ripping or tearing. The samples shall not be taken from the outer wrap of the geotextile roll nor the inner wrap of the core.

Acceptance Samples

Samples will be randomly taken by the Engineer at the job site to confirm that the geotextile meets the property values specified.

Approval will be based on testing of samples from each lot. A "lot" shall be defined for the purposes of this specification as all geotextile rolls within the consignment (*i.e.*, all rolls sent to the project site) which were produced by the same manufacturer during a continuous period of production at the same manufacturing plant and have the same product name. After the samples and manufacturer's certificate of compliance have arrived at the Headquarters Materials Laboratory in Tumwater, a maximum of 14 calendar days will be required for this testing. If the results of the testing show that a geotextile lot, as defined, does not meet the properties required for the specified use as indicated in Tables 1 through 6 the roll or rolls which were sampled will be rejected. Two additional rolls for each roll tested which failed from the lot previously tested will then be selected at random by the Engineer for sampling and retesting. If the retesting shows that any of the additional rolls tested do not meet the required properties, the entire lot will be rejected. If the test results from all the rolls retested meet the required properties, the entire lot minus the roll(s) which failed will be accepted. All geotextile which has defects, deterioration, or damage, as determined by the Engineer, will also be rejected. All rejected geotextile shall be replaced at no cost to the State.

Acceptance by Certificate of Compliance

When the quantities of geotextile proposed for use in each geotextile application are less than or equal to the following amounts, acceptance shall be by Manufacturer's Certificate of Compliance:

Application: Temporary Silt Fence   Geotextile Quantities: All quantities

The manufacturer's certificate of compliance shall include the following information about each geotextile roll to be used:

Manufacturer's name and current address,
Full product name,
Geotextile structure, including fiber/yarn type
Geotextile roll number,

Proposed geotextile use(s), and
Certified test results.

Approval of Seams
If the geotextile seams are to be sewn in the field, the Contractor shall provide a section of sewn seam before the geotextile is installed which can be sampled by the Engineer.

The seam sewn for sampling shall be sewn using the same equipment and procedures as will be used to sew the production seams. If production seams will be sewn in both the machine and cross-machine directions, the Contractor must provide sewn seams for sampling which are oriented in both the machine and cross-machine directions. The seams sewn for sampling must be at least 2 meters in length in each geotextile direction. If the seams are sewn in the factory, the Engineer will obtain samples of the factory seam at random from any of the rolls to be used. The seam assembly description shall be submitted by the Contractor to the Engineer and will be included with the seam sample obtained for testing. This description shall include the seam type, stitch type, sewing thread type(s), and stitch density.

Construction Geotextile (Installation Requirements)

Description
The Contractor shall furnish and place construction geotextile in accordance with the details shown in the Plans.

Identification, Shipment and Storage
Geotextile roll identification, storage, and handling shall be in conformance to ASTM D 4873. During periods of shipment and storage, the geotextile shall be kept dry at all times and shall be stored off the ground. Under no circumstances, either during shipment or storage, shall the material be exposed to sunlight, or other form of light which contains ultraviolet rays, for more than five calendar days.

Installation

The Contractor shall be fully responsible to install and maintain temporary silt fences at the locations shown in the Plans. A silt fence shall not be considered temporary if the silt fence must function beyond the life of the contract. The silt fence shall minimize soil carried by runoff water from going beneath, through, or over the top of the silt fence, but shall allow the water to pass through the fence. The minimum height of the top of the silt fence shall be 600 mm and the maximum height shall be 750 mm above the original ground surface. Damaged or otherwise improperly functioning portions of silt fences shall be repaired or replaced by the Contractor at no expense to the Contracting Agency, as determined by the Engineer.

The geotextile shall be attached on the up-slope side of the posts and support system with staples, wire, or in accordance with the manufacturer's recommendations. The staples or wire shall be installed through or around a 13 mm thick wood lath placed against the geotextile at the fence posts, or other method approved by the Engineer, to reduce potential for geotextile tearing at the staples or wire. Silt fence back-up support for the geotextile in the form of a wire or plastic mesh is optional, depending on the properties of the geotextile selected for use in Table 1. If wire or plastic back-up mesh is used, the mesh shall be fastened securely to the up-slope of the posts with the geotextile being up-slope of the mesh back-up support.

The geotextile shall be sewn together at all edges at the point of manufacture, or at an approved location as determined by the Engineer, to form geotextile lengths and widths as required. Alternatively, a geotextile seam may be formed by folding the geotextile from each geotextile section over on itself several times and firmly attaching the folded seam to the fence post, provided that the Contractor can demonstrate, to the satisfaction of the Engineer, that the folded geotextile seam can withstand the expected sediment loading.

The geotextile at the bottom of the fence shall be buried in a trench to a minimum depth of 150 mm below the ground surface. The trench shall be backfilled and the soil tamped in place over the buried portion of the geotextile as shown in the Plans such that no flow can pass beneath the fence nor scour occur. When wire or polymeric back-up support mesh is used, the wire or polymeric mesh shall extend into the trench a minimum of 80 mm. The fence posts shall be placed or driven a minimum of 600 mm into the ground. A minimum depth of 300 mm will be allowed if topsoil or other soft subgrade soil is not present, and the minimum depth of 600 mm cannot be reached. Fence post depths shall be increased by 150 mm if the fence is located on slopes of 3:1 or steeper and the slope is perpendicular to the fence. If the required post depths cannot be obtained, the posts shall be adequately secured by bracing or guying to prevent overturning of the fence due to sediment loading, as approved by the Engineer.

Silt fences shall be located on contour as much as possible, except at the ends of the fence, where the fence shall be turned uphill such that the silt fence captures the runoff water and prevents water from flowing around the end of the fence as shown in the Plans. If the fence must cross contours, with the exception of the ends of the fence, gravel check dams placed perpendicular to the back of the fence shall be used to minimize concentrated flow and erosion along the back of the fence. The gravel check dams shall be approximately 0.3 m deep at the back of the fence and be continued perpendicular to the fence at the same elevation until the top of the check dam intercepts the ground surface behind the fence as shown in the Plans. The gravel check dams shall consist of Crushed Surfacing Base Course (Section 9-03.9(3)), Gravel Backfill for Walls (Section 9-03.12(2)), or Shoulder Ballast (Section 9-03.9(2)). The gravel check dams shall be located every 3 m along the fence where the fence must cross contours. The slope of the fence line where contours must be crossed shall not be steeper than 3:1.

Either wood or steel posts shall be used. Wood posts shall have minimum dimensions of 40 mm by 40 mm by the minimum length shown in the Plans, and shall be free of defects such as knots, splits, or gouges. Steel posts shall consist of either size No. 8 rebar or larger, or shall consist of ASTM A 120 steel pipe with a minimum diameter of 25 mm. The spacing of the support posts shall be a maximum of 2.0 m as shown in the plans.

Fence backup support, if used, shall consist of steel wire with maximum a mesh spacing of 50 mm, or a prefabricated polymeric mesh. The strength of the wire or polymeric mesh shall be equivalent to or greater than that required in Table 1 for the geotextile (*i.e.*, 800 N grab tensile strength) if it is unsupported between posts. The polymeric mesh must be as resistant to ultraviolet radiation as the geotextile it supports.

Sediment deposits shall either be removed when the deposit reaches approximately one-third the height of the silt fence, or a second silt fence shall be installed, as determined by the Engineer.

Measurement
Construction geotextile, with the exception of temporary silt fence geotextile and underground drainage geotextile used in trench drains, will be measured by the square meter for the ground surface area actually covered. Temporary silt fence geotextile will be measured by the linear meter of silt fence installed. Underground drainage geotextile used in trench drains will be measured by the square meter for the perimeter of drain actually covered.

Payment
Payment will be made in accordance with Section 1-04.1, for each of the following bid items that are included in the
"Construction Geotextile For Temporary Silt Fence", per linear meter.
Sediment removal behind silt fences will be paid by force account under temporary water pollution/erosion control. If a new silt fence is installed in lieu of sediment removal, as determined by the Engineer, the silt fence will be paid for at the unit contract price per linear meter for "Construction Geotextile For Silt Fence".

4.5 INSTALLATION PROCEDURES

Silt fences are quite simple to construct; the normal construction sequence is shown in Figure 4-5. Installation of a prefabricated silt fence is shown is Figure 4-6.

1. Install wooden or steel fence posts or large wooden stakes in a row, with normal spacing between 0.5 to 3 m, center to center, and to a depth of 0.4 to 0.6 m. Most prefabricated fences have posts spaced approximately 2 to 3 m apart, which is usually adequate (Step 1).

2. Construct a small (minimum 0.15 m deep and 0.1 m wide) trench on the upstream side of the silt fence (Step 2).
3. Attach reinforcing wire, if required, to the posts (Step 3).
4. If a prefabricated silt fence is not being used, the geotextile must be attached to the posts using staples, reinforcing wire, or other attachments provided by the manufacturer. Geotextile should be extended at least 150 mm below the ground surface (Step 4 & 5).
5. Bury the lower end of the geotextile in the upstream trench and backfill with natural material, tamping the backfill to provide good anchorage (Step 6).

The field inspector should review the field inspection guidelines in Section 1.7.

4.6 INSPECTION AND MAINTENANCE

Silt fences should be checked periodically, especially after a rainfall or storm event. Excessive buildup of sediment must be removed so the silt fence can function properly. Generally, sediment buildup behind the fence should be removed when it reaches ⅓ to ½ of the fence height. Repair or replace any split, torn slumping or weathered geotextile. The toe trench should also be checked to be ensure that runoff is not piping under the fence.

4.7 SILT AND TURBIDITY CURTAINS

Silt and turbidity curtains perform essentially the same function as silt fences; that is, the geotextile intercepts sediment-laden water while allowing clear water to pass. Thus, for maximum efficiency, a silt or turbidity curtain should pass a maximum amount of water while retaining a maximum amount of sediment. Unfortunately, such optimum performance is normally not possible because sediments will eventually blind or clog (Figure 2-3) the geotextile. To maximize the geotextile's efficiency, the following soil, site, and environmental conditions should be established, and the geotextile selected should provide a specific filtering efficiency while maintaining the required flow rate (Bell and Hicks, 1980).

1. Grain size distribution of soil to be filtered.
2. Estimate of the soil volume to be filtered during construction.
3. Flow conditions, anticipated runoff, and water level fluctuations.
4. Expected environmental conditions, including temperature and duration of sunlight exposure.
5. Velocity, direction, and quantity of discharge water.
6. Water depth and levels of turbidity.
7. Survey of the bottom sediments and vegetation at the site.
8. Wind conditions.

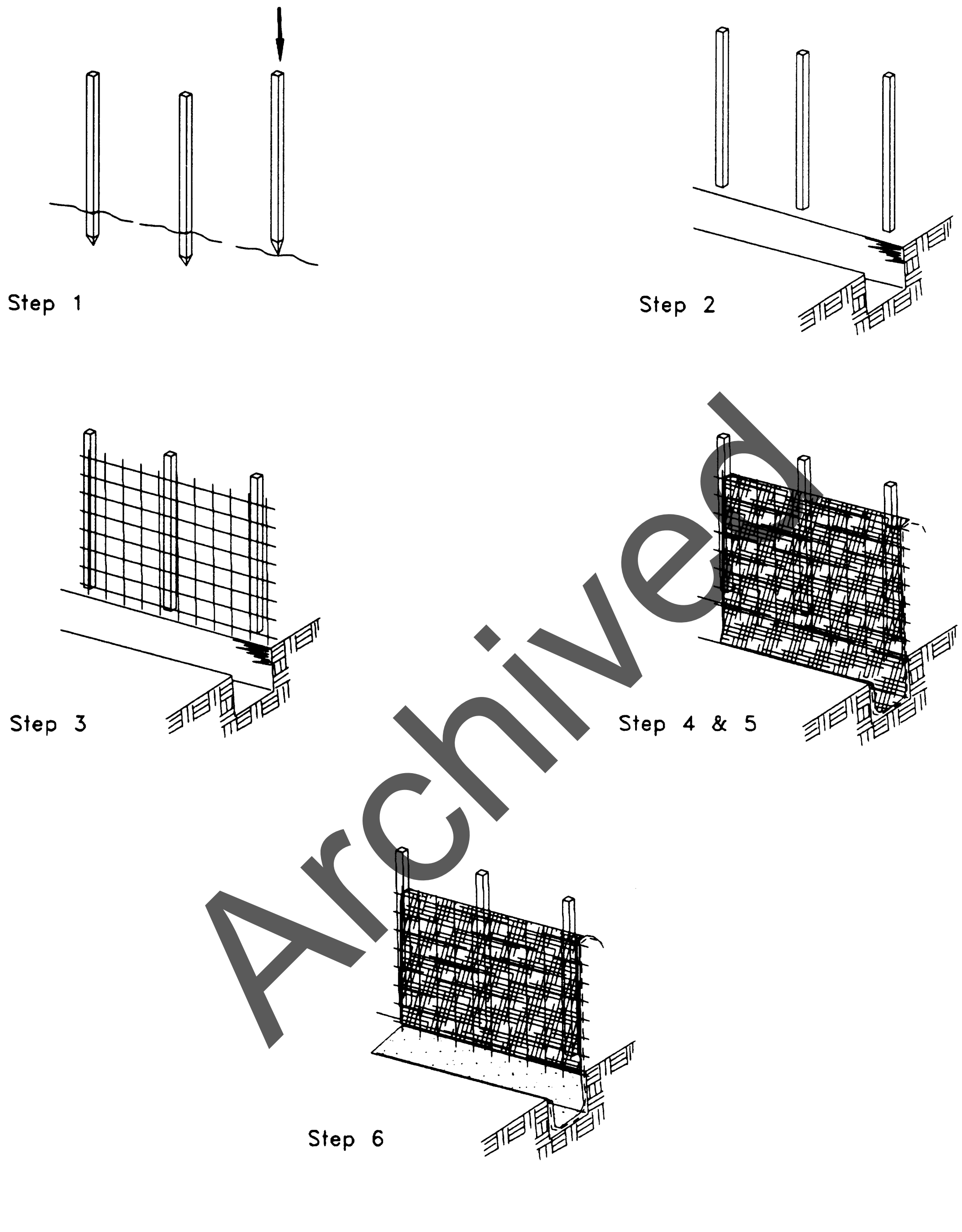


Figure 4-5 Typical silt fence installation.

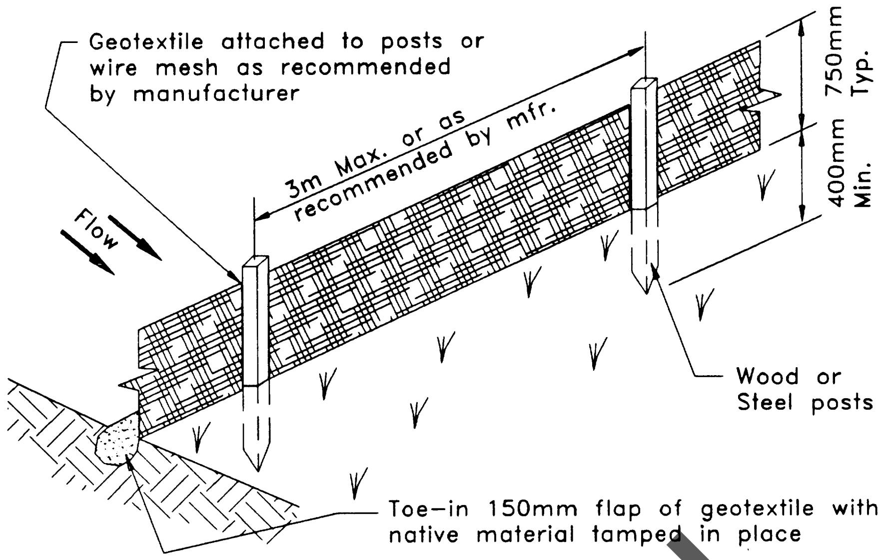


Figure 4-6 Installation of a prefabricated silt fence.

On the basis of these considerations, the geotextile can then be selected either according to the properties required to maximize particle retention and flow capacity while resisting clogging, or by performing filtration model studies such as ASTM D 5141. The first approach follows the criteria developed in Chapter 2 for drainage systems. Silt and turbidity curtains are generally concerned with fine-grained soils, therefore, the following criteria could be considered when selecting the geotextile.

A. Retention Criteria

AOS = D_{85} for woven geotextiles.
AOS = 1.8 x D_{85} for nonwovens.

NOTE: The D_{85} is a characteristic large-grain size appropriate to the suspended sediment grain size distribution. It will be strongly influenced by items Nos. 1, 3, 5, 6, and 7 above.

B. Flow Capacity Criteria

$$\psi = (10\,q) \div A$$

where:

ψ = permittivity of geotextile (T^{-1})
q = flow rate (L^3/T)

A	=	cross-sectional area silt curtain (L^2)
10	=	factor of safety

C. Clogging Resistance

Maximize AOS requirements using largest opening possible from criterion A above.

For silt and turbidity curtain construction, the geotextile forming the curtain is held vertical by flotation segments at the top and a ballast along the bottom (Bell and Hicks, 1980). A tension cable is often built into the curtain immediately above or below the flotation segments to absorb stress imposed by currents, wave action, and wind. Barrier sections are usually about 30 m long and of any required width. Curtains can also be constructed within shallow bodies of water using silt fence-type construction methods. Geotextiles have also been attached to soldier piles and draped across riprap barriers as semipermanent curtains.

The U.S. Army Corps of Engineers (1977) indicates that silt and turbidity curtains are not appropriate for certain conditions, such as:

- operations in open ocean;
- operations in currents exceeding 0.5 m/s;
- in areas frequently exposed to high winds and large breaking waves; and
- near hopper or cutter head dredges where frequent curtain movement would be necessary.

4.8 EROSION CONTROL BLANKETS

In freshly graded areas, the soil is susceptible to erosion by rainfall and runoff. Temporary, degradable blankets are used to enhance the establishment of vegetation. These products are used where vegetation alone provides sufficient site protection after the temporary products degrade. Such products are usually evaluated by field trial sections, and, therefore, are empirically designed. There are very few published records of comparative use, so the user must decide on the preferable system, usually based on local experience. You should be aware that a variety of products and systems exist. As an aid to selecting the best system, consult manufacturers and other agencies about their experiences.

Erosion protection must be provided for three distinct phases, namely:

1. prior to vegetation growth;
2. during vegetation growth; and
3. after vegetation is fully established.

Erosion control blankets provide protection during the first two phases. After vegetation is established protection can be provided by erosion control mats that reinforce the vegetation root mass, as discussed in Chapter 3.

Geosynthetic erosion control blankets are manufactured of light-weight polymer net(s) and a bedding of polymer webbing or organic materials such as straw or coconut. The bedding material protects the soil against erosion and helps retain moisture, seeds, and soil to promote growth. These polymer materials are typically not stabilized against ultraviolet light, and are designed to degrade over time. Erosion control blankets have design lives that vary between approximately 6 months to 5 years. Some blankets are provided with seeds encased in paper.

Erosion control blankets provide protection against moderate-flow velocities for short periods of time. They are typically used on moderate slopes and low velocity intermittent flow channels. Flows up to 1.5 m/s and durations of ½ to approximately 5 hours can be withstood, as illustrated in Figure 4-7. Again, design is empirical, and blanket product manufacturers should have actual flume test data and design recommendations available for their specific products. Duration of flume tests should be noted.

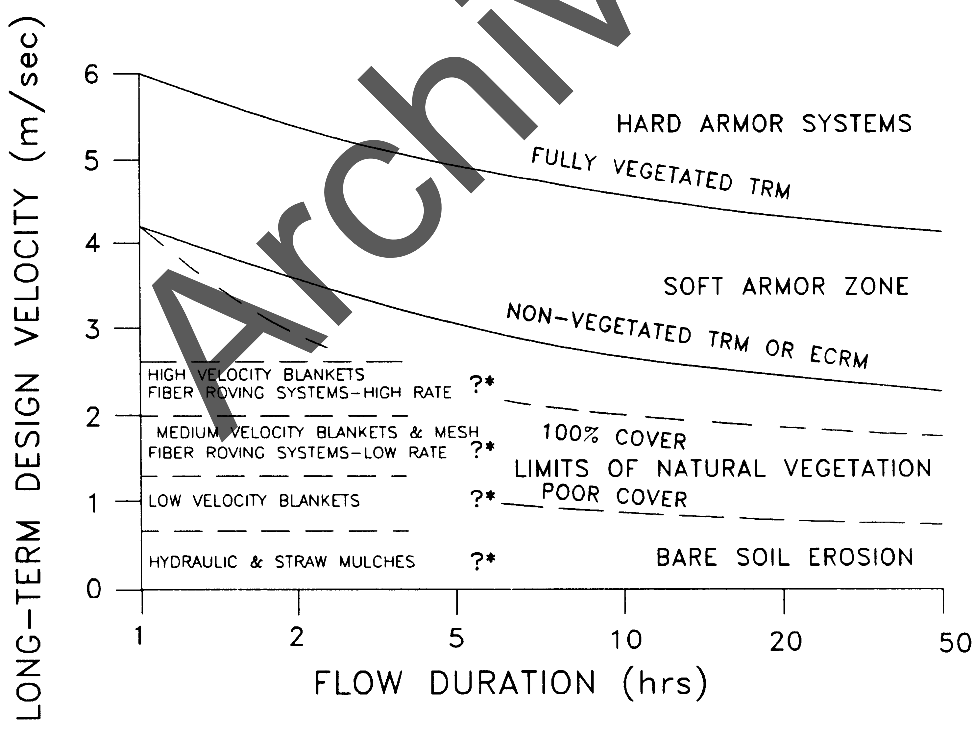


Figure 4-7 Recommended maximum design velocities and flow durations for various classes of erosion control materials (after Theisen, 1992).

Since the design of erosion control blankets is empirical, specification by index properties is not easily accomplished. Also, relatively few test methods have been standardized for erosion control blankets. Therefore, it is recommended that specifications use an *approved products* list.

Construction plans and specifications should detail and note installation requirements. Details such as anchoring in trenches, use of pins, pin length, pin spacing, roll overlap requirements, and roll termination should be addressed.

The following example specification for erosion control blankets is after the Texas Department of Transportation specification for RECP (rolled erosion control products). This agency tests candidate erosion control materials and categorizes them into classes and types in an approved materials list.

SOIL EROSION CONTROL BLANKETS

(after Texas Department of Transportation, Special Specification, Item 1225, February 1993)

1. <u>DESCRIPTION</u>.

This item shall govern for providing and placing wood, straw, or coconut fiber mat, synthetic mat, jute mesh or other material as a soil erosion control blankets on slopes or ditches or for short-term or long-term protection of seeded areas as shown on the plans or as specified by the Engineer.

2. <u>MATERIALS</u>.

(1) <u>Soil Erosion Control Blankets</u>. All soil erosion control blankets must be prequalified by the Director of Maintenance and Operations prior to use.

Prequalification procedures and a current list of prequalified materials may be obtained by writing to the Director of Maintenance and Operations. A 0.3 m x 0.3 m sample of the material may be required by the Engineer in order to verify prequalification. Samples taken, accompanied by the manufacturer's literature, will be sent, properly wrapped and identified, to the Division of Maintenance and Operations for verification.

The soil erosion control blanket shall be one (1) of the following classes and types as shown on the plans:

(a) <u>Class 1. "Slope Protection</u>"

(i) Type A. Slopes of 3:1 or flatter — Clay soils

Prequalified Type A products are:

____________	____________	____________
____________	____________	____________
____________	____________	____________

(ii) Type B. Slopes of 3:1 or flatter — Sandy soils

Prequalified Type B products are:

______________ ______________ ______________

______________ ______________ ______________

______________ ______________ ______________

(iii) Type C. Slopes steeper than 3:1 — Clay soils

Prequalified Type C products are:

______________ ______________ ______________

______________ ______________ ______________

______________ ______________ ______________

(iv) Type D. Slopes steeper than 3:1 — Sandy soils

Prequalified Type D products are:

______________ ______________ ______________

______________ ______________ ______________

______________ ______________ ______________

(b) <u>Class 2. "Flexible Channel Liner"</u>

(i) Type E. Short-term duration (Up to 2 Years)
Shear Stress (t_d) $<$ 50 Pa

Prequalified Type E products are:

______________ ______________ ______________

______________ ______________ ______________

(ii) Type F. Short-term duration (Up to 2 Years)
Shear Stress (t_d) 50 to 95 Pa

Prequalified Type F products are:

______________ ______________ ______________

______________ ______________ ______________

(2) <u>Staples</u>. Staples for anchoring the soil erosion control mat shall be U-shaped, made of 3 mm or large diameter steel wire, or other approved material, have a width of 25 to 50 mm, and a length of not less than 150 mm for firm soils and not less than 300 mm for loose soils.

3. <u>CONSTRUCTION METHODS</u>.

(1) <u>General</u>. The soil erosion control blanket shall conform to the class and type shown on the plans. The Contractor has the option of selecting an approved soil erosion control blanket conforming to the class and type shown on the plans, and according to the current approved material list.

(2) Installation. The soil erosion control blanket, whether installed as slope protection or as flexible channel liner in accordance with the approved materials list, shall be placed within 24 hours after seeding or sodding operations have been completed, or as approved by the Engineer. Prior to placing the blanket, the area to be covered shall be relatively free of all rocks or clods over 38 mm in maximum dimension and all sticks or other foreign material which will prevent the close contact of the blanket with the soil. The area shall be smooth and free of ruts or depressions exist for any reason, the Contractor shall be required to rework the soil until it is smooth and to reseed or resod the area at the Contractor's expense.

Installation and anchorage of the soil erosion control mat shall be in accordance with the project construction drawings unless otherwise specified in the contract or directed by the Engineer.

(3) Literature. The Contractor shall submit one (1) full set of manufacturer's literature and manufacturer's installation recommendations for the soil erosion control blanket selected in accordance with the approved material list.

4. MEASUREMENT.

This Item will be measured by the square meter of surface area covered.

5. PAYMENT.

The work performed and materials furnished in accordance with this Item and measured as provided under "Measurement" will be paid for at the unit price bid for "Soil Erosion Control Blanket" of the class and type shown on the plans. This price shall be full compensation for furnishing all materials, labor, tools, equipment and incidentals necessary to complete the work. Anchors, checks, terminals or junction slots, and wire staples or wood stakes will not be paid for directly but will be considered subsidiary to this Item.

4.9 REFERENCES

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5.0 GEOSYNTHETICS IN ROADWAYS AND PAVEMENTS

5.1 INTRODUCTION

The most common use of geosynthetics is in road and pavement construction. Geotextiles increase stability and improve performance of weak subgrade soils **primarily by separating the aggregate from the subgrade**. In addition, geogrids and some geotextiles can provide strength through friction or interlock developed between the aggregate and the geosynthetic. Geotextiles can also provide filtration and drainage by allowing excess pore water pressures in the subgrade to dissipate into the aggregate base course and, in cases of poor-quality aggregate, through the geotextile plane itself.

In this chapter, each of the geosynthetic functions will be discussed and related to design concepts and performance properties. Selection, specification, and construction procedures will also be presented.

5.1-1 Functions of Geosynthetics in Roadways and Pavements

A geosynthetic placed at the interface between the aggregate base course and the subgrade functions as a *separator* to prevent two dissimilar materials (subgrade soils and aggregates) from intermixing. Geotextiles and geogrids perform this function by preventing penetration of the aggregate into the subgrade (localized bearing failures) (Figure 5-1). In addition, geotextiles prevent intrusion of subgrade soils up into the base course aggregate. Localized bearing failures and subgrade intrusion occur in very soft, wet, weak subgrades. Subgrade intrusion can also occur under long term dynamic loading due to pumping and migration of fines, especially when open-graded base courses are used. It only takes a small amount of fines to significantly reduce the friction angle of select granular aggregate. Therefore, separation is important to maintain the design thickness and the stability and load-carrying capacity of the base course. Soft subgrade soils are most susceptible to disturbance during construction activities such as clearing, grubbing, and initial aggregate placement. Geosynthetics can help minimize subgrade disturbance and prevent loss of aggregate during construction. Thus, the primary function of the geotextile in this application is separation, and can in some cases be considered a secondary function for geogrids.

The system performance may also be influenced by functions of **filtration** and **drainage** (Table 1-1). The geotextile acts as a filter to prevent fines from migrating up into the aggregate due to high pore water pressures induced by dynamic wheel loads. It also acts as a drain, allowing the excess pore pressures to dissipate through the geotextile and the subgrade soils to gain strength through consolidation and improve with time.

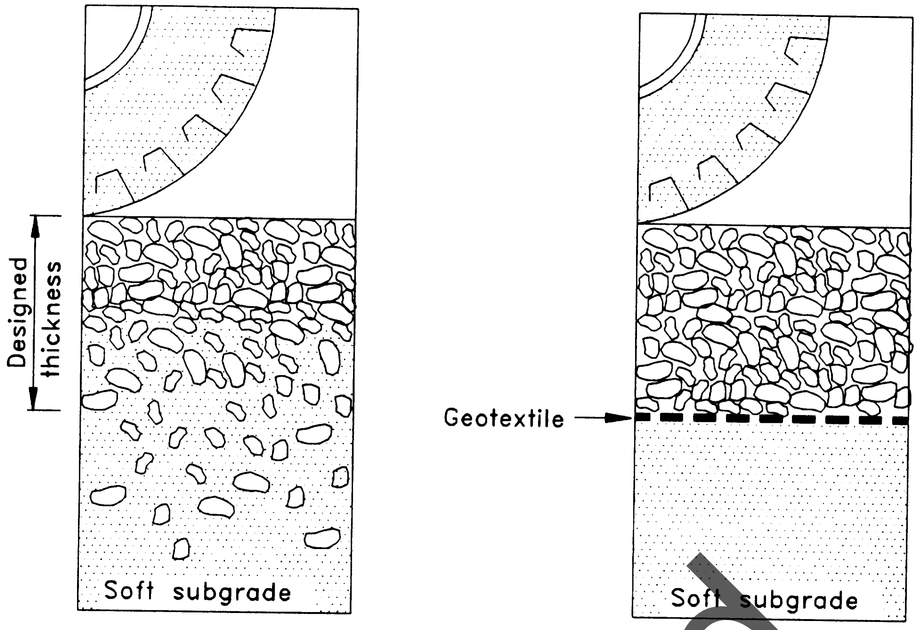


Figure 5-1 Concept of geotextile separation in roadways (after Rankilor, 1981).

System performance may also be improved through reinforcement. Geogrids and geotextiles provide reinforcement through three possible mechanisms.

1. *Lateral restraint of the base and subgrade* through friction and interlock between the aggregate, soil and the geosynthetic (Figure 5-2a).
2. *Increase in the system bearing capacity* by forcing the potential bearing capacity failure surface to develop along alternate, higher shear strength surfaces (Figure 5-2b).
3. *Membrane support* of the wheel loads (Figure 5-2c).

When an aggregate layer is loaded by a wheel or track, the aggregate tends to move or shove laterally, as shown in Figure 5-2a, unless it is restrained by the subgrade or geosynthetic reinforcement. Soft, weak subgrade soils provide very little lateral restraint, so when the aggregate moves laterally, ruts develop on the aggregate surface and also in the subgrade. A geogrid with good interlocking capabilities or a geotextile with good frictional capabilities can provide tensile resistance to lateral aggregate movement. Another possible geosynthetic reinforcement mechanism is illustrated in Figure 5-2b. Using the analogy of a wheel load to a footing, the geosynthetic reinforcement forces the potential bearing capacity failure surface to follow an alternate higher strength path. This tends to increase the bearing capacity of the roadway.

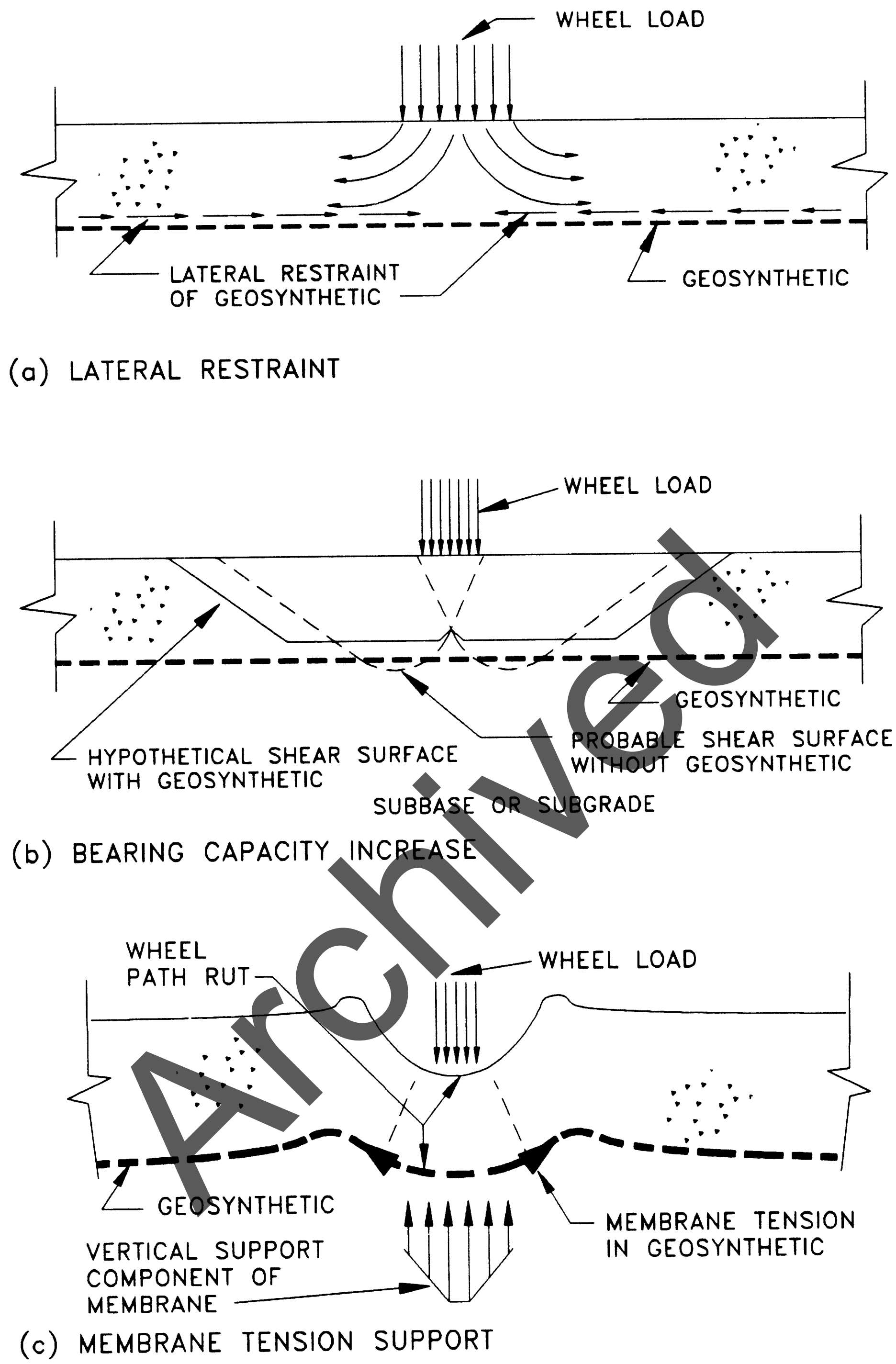


Figure 5-2 Possible reinforcement functions provided by geosynthetics in roadways: (a) lateral restraint, (b) bearing capacity increase, and (c) membrane tension support (after Haliburton, et al., 1981).

A third possible geosynthetic reinforcement function is membrane-type support of wheel loads, as shown conceptually in Figure 5-2c. In this case, the wheel load stresses must be great enough to cause plastic deformation and ruts in the subgrade. If the geosynthetic has a sufficiently high tensile modulus, tensile stresses will develop in the reinforcement, and the vertical component of this membrane stress will help support the applied wheel loads. As tensile stress within the geosynthetic cannot be developed without some elongation, wheel path rutting (in excess of 100 mm) is required to develop membrane-type support. Therefore, this mechanism is generally limited to temporary roads or the first aggregate lift in permanent roadways.

5.1-2 Subgrade Conditions in which Geosynthetics are Useful

Geotextile separators have a 20+ year history of successful use for the stabilization of very soft wet subgrades. Based on experience and several case histories summarized by Haliburton, Lawmaster, and McGuffey (1981) and Christopher and Holtz (1985), the following subgrade conditions are considered to be the most appropriate for geosynthetic use in roadway construction:

- Poor soils
  (USCS: SC, CL, CH, ML, MH, OL, OH, and PT)
  (AASHTO: A-5, A-6, A-7-5, and A-7-6)
- Low undrained shear strength
  $\tau_f = c_u < 90$ kPa
  CBR < 3 {Note: CBR as determined with ASTM D 4429 Bearing Ratio of Soils in Place (1994)}
  $M_R \approx 30$ MPa
- High water table
- High sensitivity

Under these conditions, geosynthetics function primarily as separators and filters to stabilize the subgrade, improving construction conditions and allowing long-term strength improvements in the subgrade. If large ruts develop during placement of the first aggregate lift, then some reinforcing effect is also present. As a summary recommendation, the following geotextile functions are appropriate for the corresponding subgrade strengths:

Undrained Shear Strength (kPa)	Subgrade CBR	Functions
60 - 90	2 - 3	Filtration and possibly separation
30 - 60	1 - 2	Filtration, separation, and possibly reinforcement
< 30	< 1	All functions, including reinforcement

As the geosynthetic allows for subgrade improvement with time, AASHTO M288 has identified applications where the undrained shear strength is less than about 90 kPa (CBR about 3) as *stabilization* applications. From a foundation engineering point of view, clay soils with undrained shear strengths of 90 kPa are considered to be stiff clays (Terzaghi and Peck, 1967, p 30) and are generally quite good foundation materials. Allowable footing pressures on such soils equal 150 kPa or greater. Simple stress distribution calculations show that for **static** loads, such soils will readily support reasonable truck loads and tire pressures, even under relatively thin granular bases.

Dynamic loads and high tire pressures are another matter. Some rutting will probably occur in such soils, especially after a few hundred passes (Webster, 1993). If traffic is limited, as it is in many temporary roads, or if shallow (< 75 mm) ruts are acceptable, as in most construction operations, then a maximum undrained shear strength of about 90 kPa (CBR = 3) for geosynthetic use in highway construction seems reasonable. However, for soils that are seasonally weak (*e.g.*, from frost heave) or for high fines content soils which are susceptible to pumping, a geotextile separator may be of benefit in preventing migration of fines. This is especially the case for permeable base applications. Even on firm subgrades, a geotextile placed beneath the base functions as a separator and filter, as illustrated in Figure 5-3. A greater range of geotextile applicability is recognized in the M288 specification (AASHTO, 1997) with a CBR ≥ 3 the geotextile application is identified as separation. Further discussion of potential applicability of geotextiles on soils with CBR > 3 is presented in Appendix G of this manual, and the complete M288 specification is presented in Appendix D.

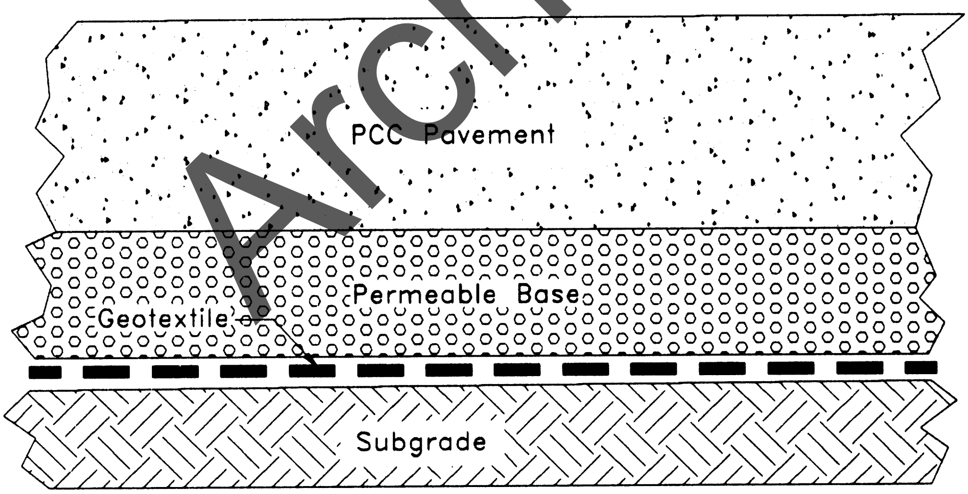


Figure 5-3 Geotextile separator beneath permeable base (Baumgartner, 1994).

5.2 APPLICATIONS

5.2-1 Temporary and Permanent Roads

Roads and highways are broadly classified into two categories: permanent and temporary, depending on their service life, traffic applications, or desired performance. Permanent roads include both paved and unpaved systems which usually remain in service 10 years or more. Permanent roads may be subjected to more than a million load applications during their design lives. On the other hand, temporary roads are, in most cases, unpaved. They remain in service for only short periods of time (often less than 1 year), and are usually subjected to fewer than 10,000 load applications during their services lives. Temporary roads include detours, haul and access roads, construction platforms, and stabilized working *tables* required for the construction of permanent roads, as well as embankments over soft foundations.

Geosynthetics allow construction equipment access to sites where the soils are normally too weak to support the initial construction work. This is one of the more important uses of geosynthetics. Even if the finished roadway can be supported by the subgrade, it may be virtually impossible to begin construction of the embankment or roadway. Such sites require stabilization by dewatering, demucking, excavation and replacement with select granular materials, utilization of stabilization aggregate, chemical stabilization, etc. Geosynthetics can often be a cost-effective alternate to these expensive foundation treatment procedures.

Furthermore, geosynthetic separators enable contractors to meet minimum compaction specifications for the first two or three aggregate lifts. This is especially true on very soft, wet subgrades, where the use of ordinary compaction equipment is very difficult or even impossible. Long term, a geosynthetic acts to maintain the roadway design section and the base course material integrity. Thus, the geosynthetic will ultimately increase the life of the roadway.

5.2-2 Benefits

Geosynthetics used in roadways on soft subgrades, may provide several cost and performance benefits, including the following.

1. Reducing the intensity of stress on the subgrade and preventing the base aggregate from penetrating into the subgrade **(function: separation)**.
2. Preventing subgrade fines from pumping or otherwise migrating up into the base **(function: separation and filtration)**.
3. Preventing contamination of the base materials which may allow more open-graded, free-draining aggregates to be considered in the design **(function: filtration)**.
4. Reducing the depth of excavation required for the removal of unsuitable subgrade materials

(function: separation and reinforcement).

5. Reducing the thickness of aggregate required to stabilize the subgrade **(function: separation and reinforcement).**
6. Reducing disturbance of the subgrade during construction **(function: separation and reinforcement).**
7. Allowing an increase in subgrade strength over time **(function: filtration).**
8. Reducing the differential settlement of the roadway, which helps maintain pavement integrity and uniformity **(function: reinforcement).** Geosynthetics will also aid in reducing differential settlement in transition areas from cut to fill. {NOTE: Total and consolidation settlements are not reduced by the use of geosynthetic reinforcement.}
9. Reducing maintenance and extending the life of the pavement **(functions: all).**

Geosynthetics are also used in permanent roadways to provide capillary breaks to reduce frost action in frost-susceptible soils, and to provide membrane-encapsulated soil layers (MESL) to reduce the effects of seasonal water content changes on roadways on swelling clays.

5.3 POSSIBLE FAILURE MODES OF PERMANENT ROADS

Yoder and Witczak (1975) define two types of pavement distress, or failure. The first is a structural failure, in which a collapse of the entire structure or a breakdown of one or more of the pavement components renders the pavement incapable of sustaining the loads imposed on its surface. The second type failure is a functional failure; it occurs when the pavement, due to its roughness, is unable to carry out its intended function without causing discomfort to drivers or passengers or imposing high stresses on vehicles. The cause of these failure conditions may be due to excessive loads, climatic and environmental conditions, poor drainage leading to poor subgrade conditions, and disintegration of the component materials. Excessive loads, excessive repetition of loads, and high tire pressures can cause either structural or functional failures.

Pavement failures may occur due to the intrusion of subgrade soils into the granular base, which results in inadequate drainage and reduced stability. Distress may also occur due to excessive loads that cause a shear failure in the subgrade, base course, or the surface. Other causes of failures are surface fatigue and excessive settlement, especially differential of the subgrade. Volume change of subgrade soils due to wetting and drying, freezing and thawing, or improper drainage may also cause pavement distress. Inadequate drainage of water from the base and subgrade is a major cause of pavement problems (Cedergren, 1987). If the subgrade is saturated, excess pore pressures will develop under traffic loads, resulting in subsequent softening of the subgrade. Under dynamic loading, fines can be literally pumped up into the subgrade or base.

Improper construction practices may also cause pavement distress. Wetting of the subgrade during construction may permit water accumulation and subsequent softening of the subgrade in the rutted areas after construction is completed. Use of dirty aggregates or contamination of the base aggregates during construction may produce inadequate drainage, instability, and frost susceptibility. Reduction in design thickness during construction due to insufficient subgrade preparation may result in undulating subgrade surfaces, failure to place proper layer thicknesses, and unanticipated loss of base materials due to subgrade intrusion. Yoder and Witczak (1975) state that a major cause of pavement deterioration is inadequate observation and field control by qualified engineers and technicians during construction.

After construction is complete, improper or inadequate maintenance may also result in pavement distress. Sealing of cracks and joints at proper intervals must be performed to prevent surface water infiltration. Maintenance of shoulders will also affect pavement performance.

As indicated in the list of possible benefits resulting from geosynthetic use in permanent roadway systems (section 5.2-2), properly designed geosynthetics can enhance pavement performance and reduce the likelihood of failures.

5.4 ROADWAY DESIGN USING GEOTEXTILES

Certain design principles are common to all types of roadways, regardless of the design method. Basically, the design of any roadway involves a study of each of the components of the system, (surface, aggregate base courses and subgrade) detailing their behavior under traffic load and their ability to carry that load under various climatic and environmental conditions. All roadway systems, whether permanent or temporary, derive their support from the underlying subgrade soils. Thus, the geotextile functions are similar for either temporary or permanent roadway applications. However, due to different performance requirements, **design methodologies for temporary roads should not be used to design permanent roads**. Temporary roadway design usually allows some rutting to occur over the design life, as ruts will not necessarily impair service. Obviously, ruts are not acceptable in permanent roadways. In the following two sections, recommended design procedures for both temporary and permanent roads are presented. Our permanent road and pavement design basically uses geotextiles for the construction or stabilization lift only; the base course thickness required to adequately carry the design traffic loads for the design life of the pavement is not reduced due to the use of a geotextile. There is some evidence, however, that suggests a geogrid placed at the bottom of the aggregate base may permit a 10 to 20% base thickness reduction, as noted in Appendix G, Recent Roadway Research.

5.5 GEOTEXTILE SURVIVABILITY

Selecting a geotextile for either permanent or temporary roads depends upon one thing -- the survivability criteria. If the roadway system is designed correctly, then the stress at the top of the subgrade due to the weight of the aggregate and the traffic load should be less than the bearing capacity of the soil plus a safety factor. However, the stresses applied to the subgrade and the geotextile during construction may be much greater than those applied in service. **Therefore, selection of the geotextile in roadway applications is usually governed by the anticipated construction stresses.** This is the concept of geotextile survivability -- the geotextile must survive the construction operations if it is to perform its intended function.

The geotextile strength required to survive the most severe conditions anticipated during construction is listed in Table 5-1 (a Class 1 geotextile per AASHTO M288 (1997)). Geotextiles that meet or exceed these survivability requirements can be considered acceptable for most projects. The selected geotextile must also retain the underlying subgrade soils, allowing the subgrade to drain freely, consolidate, and gain strength. Thus, the geotextile must be checked, using the drainage and filtration requirements discussed in Chapter 2. Default geotextile requirements are presented in Table 5-1.

The survivability requirements in Table 5-1 were based on both research and on the properties of geotextiles which have performed satisfactorily as separators in roads and in similar applications. In the absence of any other information, they should be used as **minimum** property values. Judgment and experience may be used to reduce the geotextile requirements as indicated by AASHTO M288.

Geotextiles with less survivability strength (*i.e.*, a Class 2 geotextile per AASHTO M288 (1997)) may be acceptable for applications where a *moderate* level of survivability is needed. Table 5-2 relates the elements of construction (*i.e.*, equipment, aggregate characteristics, subgrade preparation, and subgrade shear strength) to the severity of the loading imposed on the geotextile. If one or more of these items falls within a particular severity category (*i.e.*, moderate or high), then geotextiles meeting those survivability requirements should be selected. A Class 1 geotextile should be used for the high category, and a Class 2 geotextile may be considered for the moderate category. Variable combinations indicating a NOT RECOMMENDED rating suggests that one or more variables should be modified to assure a successful installation. Some judgment is required in using these criteria.

TABLE 5-1
GEOTEXTILE PROPERTY REQUIREMENTS[1,2,3]
FOR GEOTEXTILES IN STABILIZATION APPLICATIONS
(after AASHTO, 1997)

Property	ASTM Test Method	Units	Requirement	
SURVIVABILITY			Geotextile Class 1[4]	
			Elongation < 50%[5]	Elongation > 50%[5]
Grab Strength	D 4632	N	1400	900
Sewn Seam Strength[6]	D 4632	N	1200	810
Tear Strength	D 4533	N	500	350
Puncture Strength	D 4833	N	500	350
Burst Strength	D 3786	kPa	3500	1700
Ultraviolet Stability (Retained Strength)	D 4355	%	50% after 500 hours of exposure	
DRAINAGE AND FILTRATION[7]				
Apparent Opening Size	D 4751	mm	< 0.6 for < 50% passing 0.075 mm sieve < 0.3 for > 50%passing 0.075 mm sieve	
Permittivity	D 4491	sec^{-1}	0.5 for < 15% passing 0.075 mm sieve 0.2 for 15 to 50% passing 0.075 mm sieve 0.1 for > 50% passing 0.075 mm sieve	

NOTES:
1. Acceptance of geotextile material shall be based on ASTM D 4759.
2. Acceptance shall be based upon testing of either conformance samples obtained using Procedure A of ASTM D 4354, or based on manufacturer's certifications and testing of quality assurance samples obtained using Procedure B of ASTM D 4354.
3. Minimum; use value in weaker principal direction. All numerical values represent minimum average roll value (*i.e.*, test results from any sampled roll in a lot shall meet or exceed the minimum values in the table). Lot samples according to ASTM D 4354.
4. Default geotextile selection. The engineer may specify a Class 2 geotextile (see Appendix D) for moderate survivability conditions, see Table 5-2.
5. As measured in accordance with ASTM D 4632.
6. When seams are required. Values apply to both field and manufactured seams.
7. Also, the geotextile permeability should be greater than the soil permeability.

TABLE 5-2
CONSTRUCTION SURVIVABILITY RATINGS
(after Task Force 25, 1990)

Site Soil CBR at Installation[1]	< 1		1 to 2		> 3	
Equipment Ground Contact Pressure (kPa)	> 350	< 350	> 350	< 350	> 350	< 350
Cover Thickness[2] (compacted, mm)						
100[3,4]	NR[5]	NR	1[5]	1	2[5]	2
150	NR	NR	1	1	2	2
300	NR	1	2	2	2	2
450	1	2	2	2	2	2

NOTES:
1. Assume saturated CBR unless construction scheduling can be controlled.
2. Maximum aggregate size not to exceed one-half the compacted cover thickness.
3. For low-volume, unpaved roads (ADT < 200 vehicles).
4. The 100 mm minimum cover is limited to existing road bases and is not intended for use in new construction.
5. NR = NOT RECOMMENDED. High survivability relates to a Class 1 geotextile and Moderate survivability relates to a Class 2 geotextile, per AASHTO M288 (1997).

Survivability of geogrids and geotextiles for major projects should be verified by conducting field tests under site-specific conditions. These field tests should involve trial sections using several geosynthetics on typical subgrades at the project site and implementing various types of construction equipment. After placement of the geosynthetics and aggregate, the geosynthetics are exhumed to see how well or how poorly they tolerated the imposed construction stresses. These tests could be performed during design or after the contract was let, similar to the recommendations for riprap placement (Section 3.8-1). In the latter case, the contractor is required to demonstrate that the proposed subgrade condition, equipment, and aggregate placement will not damage the geotextile or geogrid. If necessary, additional subgrade preparation, increased lift thickness, and/or possibly different construction equipment could be utilized. In rare cases, the contractor may even have to supply a different geosynthetic.

5.6 DESIGN GUIDELINES FOR TEMPORARY AND UNPAVED ROADS

There are two main approaches to the design of temporary and unpaved roads. The first assumes no reinforcing effect of the geotextile; that is, the geotextile acts as a separator only. The second approach considers a possible reinforcing effect due to the geotextile. It appears that the separation function is more important for thin roadway sections with relatively small live loads where ruts, approximating 50 to 100 mm are anticipated. In these cases, a design which assumes no reinforcing effect is generally conservative. On the other hand, for large live loads on thin roadways where deep ruts (> 100 mm) may occur, and for thicker roadways on softer subgrades, the reinforcing function becomes increasingly more important if stability is to be maintained. It is for these latter cases that reinforcing analyses have been developed and are appropriate.

The design method presented in this manual considers mainly the separation and filtration functions. It was selected because it has a long history of successful use, it is based on principles of soil mechanics, and it has been calibrated by full-scale field tests. It can also be adapted to a wide variety of conditions. Other methods considering reinforcement functions are described by Koerner (1994), Christopher and Holtz (1985) and Giroud and Noiray (1981). For roadways where stability of the embankment foundation is questionable (*i.e.*, $(\gamma H)/c > 3$), refer to Chapter 7 for information on reinforced embankments.

The following design method was developed by Steward, Williamson, and Mohney (1977) for the U.S. Forest Service (USFS). It allows the designer to consider:

- vehicle passes;
- equivalent axle loads;
- axle configurations;
- tire pressures;
- subgrade strengths; and
- rut depths.

The following limitations apply:

- the aggregate layer must be
  a) compacted to CBR 80,
  b) cohesionless (nonplastic);
- vehicle passes less than 10,000;
- geotextile survivability criteria must be considered; and
- subgrade undrained shear strength less than about 90 kPa (CBR < 3).

As discussed in Section 5.1-2, for subgrades stronger than about 90 kPa (CBR > 3), geotextiles are rarely required for stabilization, although they may provide some drainage and filtration. In this case, the principles developed in Chapter 2 are applicable, just as they are for weaker subgrades where drainage and filtration are likely to be very important.

Based on both theoretical analysis and empirical (laboratory and full-scale field) tests on geotextiles, Steward, Williamson and Mohney (1977) determined that a certain amount of rutting would occur under various traffic conditions, both with and without a geotextile separator and for a given stress level acting on the subgrade. They present this stress level in terms of bearing capacity factors, similar to those commonly used for the design of shallow foundations on cohesive soils. These factors and conditions are given in Table 5-3.

TABLE 5-3
BEARING CAPACITY FACTORS FOR DIFFERENT RUTS AND TRAFFIC CONDITIONS BOTH WITH AND WITHOUT GEOTEXTILE SEPARATORS
(after Steward, Williamson, and Mohney, 1977)

Condition	Ruts (mm)	Traffic (Passes of 80 kN axle equivalents)	Bearing Capacity Factor, N_c
Without Geotextile	<50 >100	>1000 <100	2.8 3.3
With Geotextile	<50 >100	>1000 <100	5.0 6.0

The following design procedure is recommended:

STEP 1. Determine soil subgrade strength.

Determine the subgrade soil strength in the field using the field CBR, cone penetrometer, vane shear, resilent modulus, or any other appropriate test. The undrained shear strength of the soil, c, can be obtained from the following relationships:

- for field CBR, c in kPa = 30 x CBR;
- for the WES cone penetrometer, c = cone index divided by 10 or 11; and
- for the vane shear test, c is directly measured.

Other in-situ tests, such as the static cone penetrometer test (CPT) or dilatometer (DMT), may be used, provided local correlations with undrained shear strength exist. Use of the Standard Penetration Test (SPT) is **not** recommended for soft clays.

STEP 2. Determine subgrade strength at several locations and at different times of the year.

Make strength determinations at several locations where the subgrade appears to be the weakest. Strengths should be evaluated at depth of 0 to 200 mm and from 200 - 500 mm; six to ten strength measurements are recommended at each location to obtain a good average value. Tests should also be performed when the soils are in their weakest condition, when the water table is the highest, etc.

STEP 3. Determine wheel loading.

Determine the maximum single wheel load, maximum dual wheel load, and the maximum dual tandem wheel load anticipated for the roadway during the design period. For example, an 8 m^3 dump truck with tandem axles will have a dual wheel load of approximately 35 kN. A motor grader has a wheel load of 22 to 44 kN.

STEP 4. Estimate amount of traffic.

Estimate the maximum amount of traffic anticipated for each design vehicle class.

STEP 5. Establish tolerable rutting.

Establish the amount of tolerable rutting during the design life of the roadway. For example, 50 to 75 mm of rutting is generally acceptable during construction.

STEP 6. Obtain bearing capacity factor.

Obtain appropriate subgrade stress level in terms of the bearing capacity factors in Table 5-3.

STEP 7. Determine required aggregate thickness.

Determine the required aggregate thickness from the USFS design charts (Figures 5-4, 5-5, and 5-6) for each maximum loading. Enter the curve with appropriate bearing capacity factors (N_c) multiplied by the design subgrade undrained shear strength (c) to evaluate each required stress level (cN_c).

STEP 8. Select design thickness.

Select the design thickness based on the design requirements. The design thickness should be given to the next higher 25 mm.

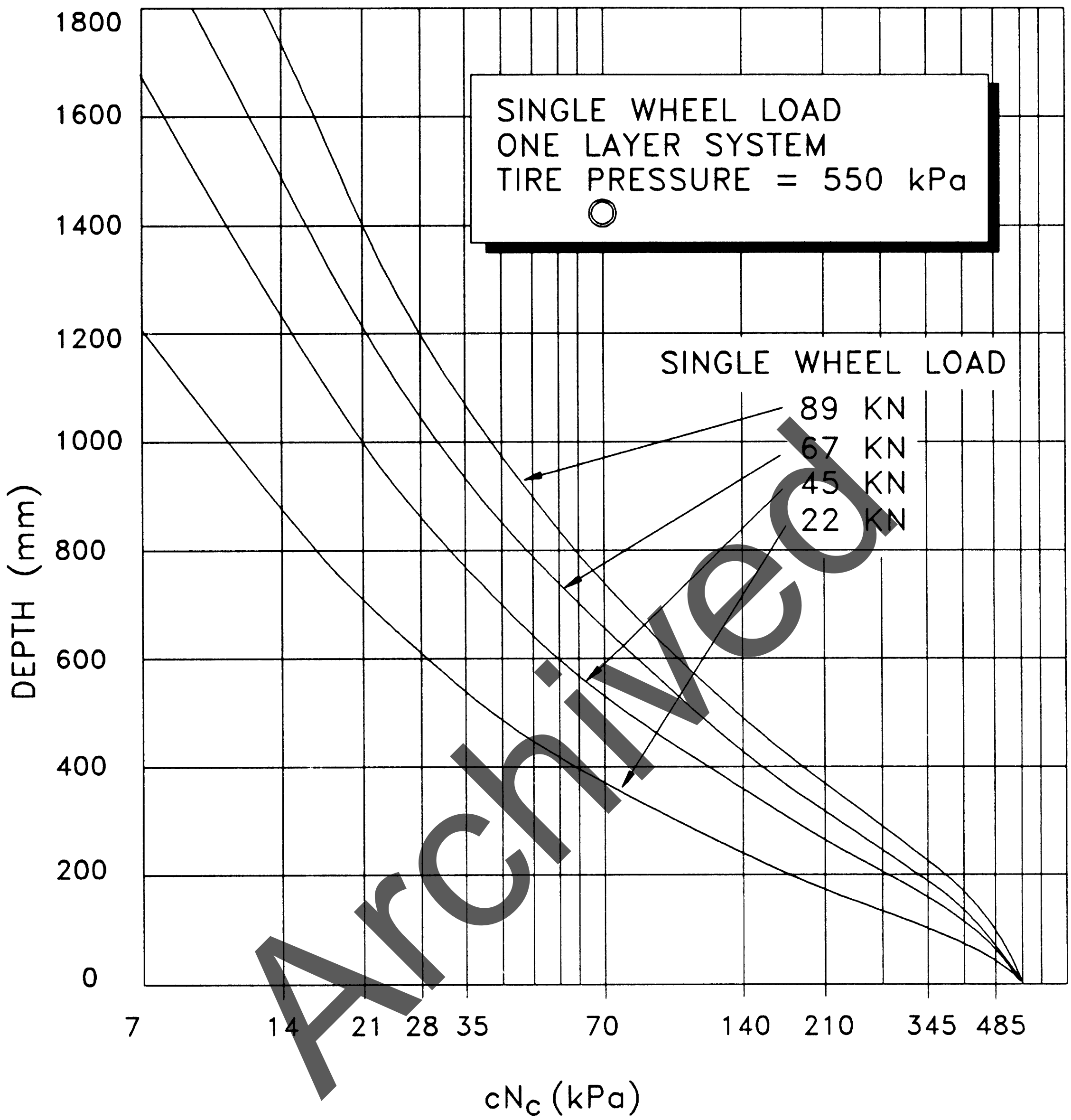


Figure 5-4 U.S. Forest Service thickness design curve for single wheel load (Steward et al., 1977).

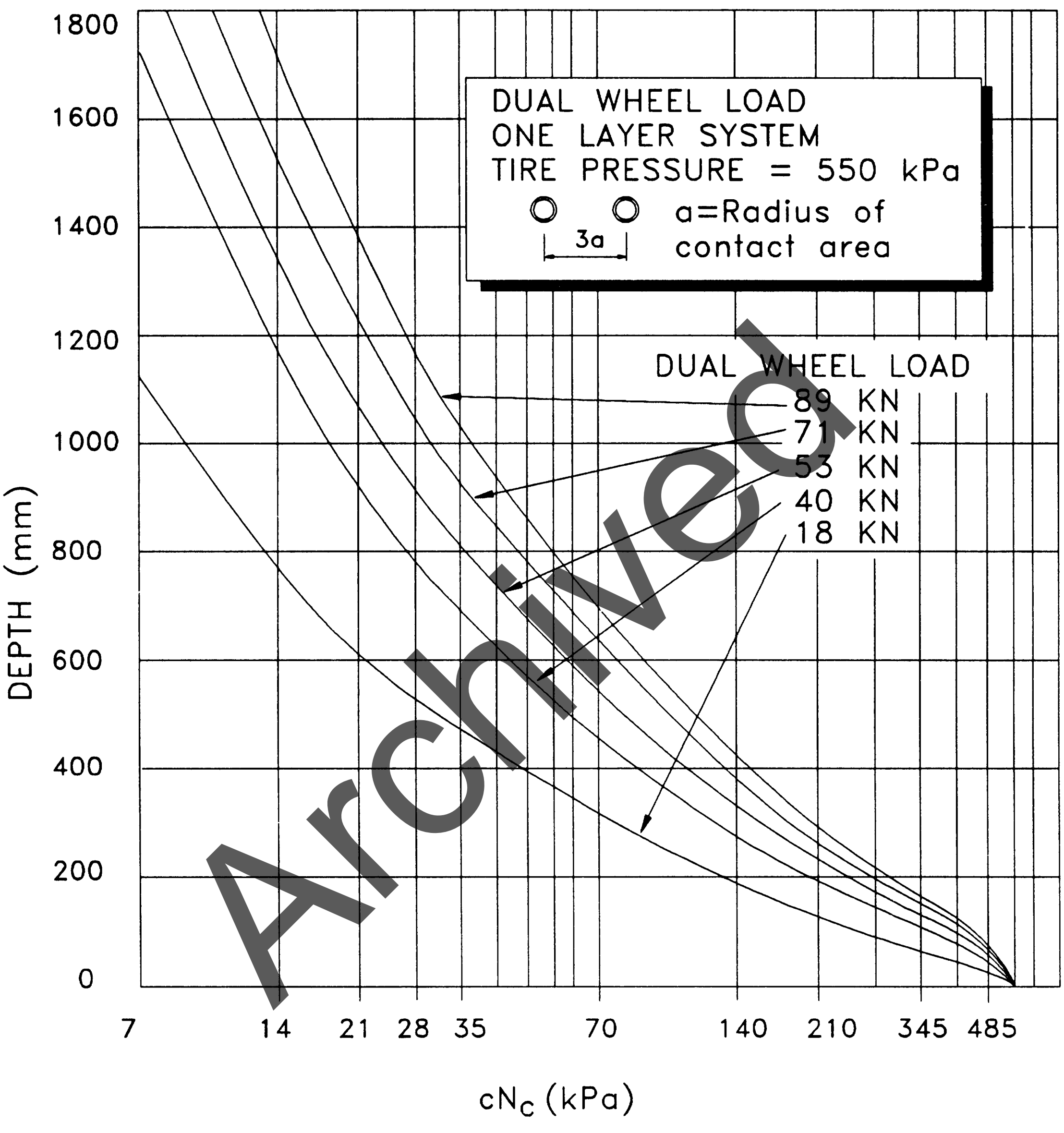


Figure 5-5 U.S. Forest Service thickness design curve for dual wheel load (Steward et al., 1977).

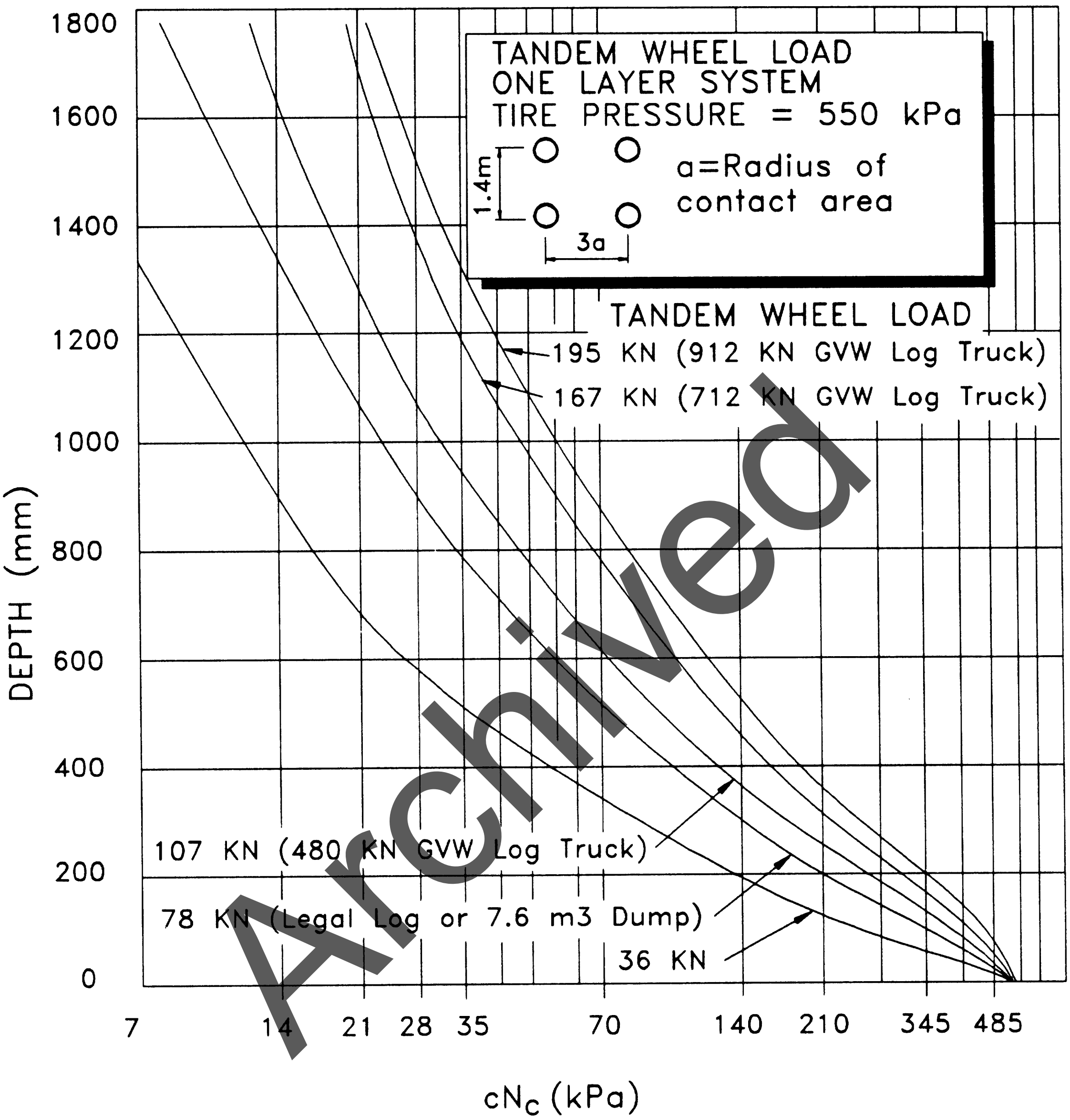


Figure 5-6 U.S. Forest Service thickness design curve for tandem wheel load (Steward et al., 1977).

STEP 9. Check geotextile drainage and filtration characteristics.

Check the geotextile drainage and filtration requirements. Use the gradation and permeability of the subgrade, the water table conditions, and the retention and permeability criteria given in Chapter 2. In high water table conditions with heavy traffic, filtration criteria may also be required. From Chapter 2, that criteria is:

$AOS \leq D_{85}$ (Wovens) (Eq. 2-3)

$AOS \leq 1.8\ D_{85}$ (Nonwovens) (Eq. 2-4)

$k_{geotextile} \geq k_{soil}$ (Eq. 2-7a)

$\psi \geq 0.1\ sec^{-1}$ (Eq. 2-8c)

STEP 10. Determine geotextile survivability requirements.

Check the geotextile survivability strength requirements as discussed in Section 5.5.

STEP 11. Specify geotextile property requirements.

Specify geotextiles that meet or exceed these survivability criteria.

STEP 12. Specify construction requirements.

Follow the construction recommendations in Section 5.12

5.7 TEMPORARY ROAD DESIGN EXAMPLE

DEFINITION OF DESIGN EXAMPLE

- Project Description: A haul road over wet, soft soils is required for a highway construction project.
- Type of Structure: temporary unpaved road
- Type of Application: geotextile separator
- Alternatives:
  i) excavate unsuitable material and increased aggregate thickness
  ii) geotextile separator between aggregate and subgrade
  iii) use an estimated depth of aggregate and maintain as required

GIVEN DATA

- subgrade
  - cohesive subgrade soils
  - high water table
  - average undrained shear strength about 30 kPa or CBR = 1
- traffic
  - approximately 5000 passes
  - 90 kN single axle truck
  - 550 kPa tire pressure

- ruts - maximum of 50 to 100 mm

REQUIRED

Design the roadway section.

Consider: 1) design without a geotextile; and 2) alternate with geotextile.

DEFINE

A. Geotextile function(s):

B. Geotextile properties required:

C. Geotextile specification:

SOLUTION

A. Geotextile function(s):
Primary - separation
Secondary - filtration, drainage, reinforcement

B. Geotextile properties required:
survivability
apparent opening size (AOS)

DESIGN Design roadway with and without geotextile inclusion. Compare options.

STEP 1. DETERMINE SOIL SUBGRADE STRENGTH

given - CBR ≈ 1

STEP 2. DETERMINE SUBGRADE STRENGTH AT SEVERAL LOCATIONS

Assume that CBR ≈ 1 is taken from area(s) where the subgrade appears to be the weakest.

STEP 3. DETERMINE WHEEL LOADING

given - 90 kN single-axle truck, with 550 kPa tire pressure
- therefore, 45 kN single wheel load

STEP 4. ESTIMATE AMOUNT OF TRAFFIC

given - 5,000 passes

STEP 5. ESTABLISH TOLERABLE RUTTING

given - 150 to 200 mm

STEP 6. OBTAIN BEARING CAPACITY FACTOR

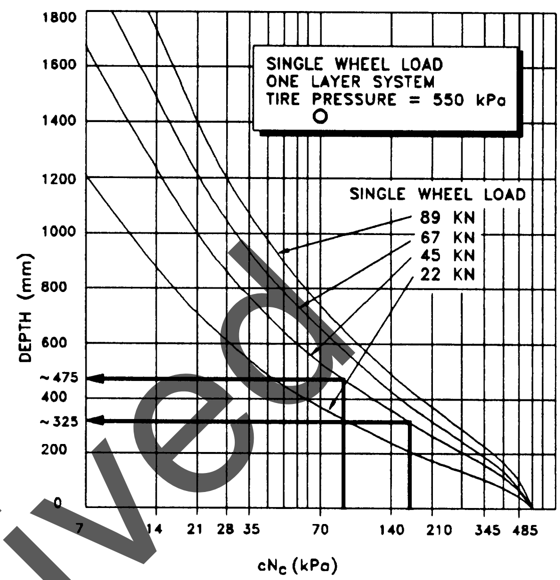
without a geotextile:
- $2.8 < N_c < 3.3$
- assume $N_c \approx 3.0$ for 5,000 passes and 50 to 100 mm ruts

with a geotextile:
- $5.0 < N_c < 6.0$
- assume $N_c \approx 5.5$ for 5,000 passes and 50 to 100 mm ruts

STEP 7. DETERMINE REQUIRED AGGREGATE THICKNESSES

without a geotextile
- $c\ N_c = 30\ \text{kPa} \times 3.0 = 90\ \text{kPa}$
- depth of aggregate ≈ 475 mm

with a geotextile
- $c\ N_c = 30\ \text{kPa} \times 5.5 = 165\ \text{kPa}$
- depth of aggregate ≈ 325 mm



STEP 8. SELECT DESIGN THICKNESS

Use 325 mm and a geotextile

STEP 9. CHECK GEOTEXTILE DRAINAGE AND FILTRATION CHARACTERISTICS

Use AOS < 0.3 mm and permittivity ≥ 0.1 sec^{-1}, per requirement of Table 5-1 since soil has > 50% passing the 0.075 mm sieve. Permeability of geotextile must be greater than soil permeability.

STEP 10. DETERMINE GEOTEXTILE SURVIVABILITY REQUIREMENTS

Use Table 5-2: with CBR = 1, dump truck contact pressure > 550 kPa, and 325 mm cover thickness, and find a **MODERATE** survivability to **NOT RECOMMENDED** rating.

Use a HIGH, or Class 1, survivability geotextile, or greater.

STEP 11. SPECIFY GEOTEXTILE PROPERTY REQUIREMENTS

From Table 5-1; geotextile separator shall meet or exceed the minimum average roll values, with elongation at failure determined with the ASTM D 4632 test method, of:

Property	ASTM Test Method	Elongation < 50%	Elongation > 50%
Grab Strength	D 4632	1400	900
Sewn Seam Strength	D 4632	1200	810
Tear Resistance	D 4533	500	350

Puncture	D 4833	500	350
Burst	D 3786	3500	1700
Ultraviolet Stability	D 4355	50% strength retained after 500 hours	

The geotextile shall have an AOS < 0.3 mm, $\psi \geq 0.1 \text{ sec}^{-1}$, and the permeability shall be ______.

STEP 12. SPECIFY CONSTRUCTION REQUIREMENTS

See Section 5.12

5.8 DESIGN GUIDELINES FOR PERMANENT ROADWAYS

The recommended design method for using geotextiles in permanent pavements is that developed by Christopher and Holtz (1985; 1991). It is based on the following concepts:

1. Standard methods are used to design the overall pavement system (*i.e.*, AASHTO, CBR, R-value, resilent modulus, etc.).
2. **The geotextile is assumed to provide no structural support, therefore, no reduction is allowed in aggregate thickness required for structural support.**
3. Aggregate savings is achieved through a reduction in the stabilization aggregate required for construction but not used for structural support.
4. The recommended method is used to design the first construction lift, which is called the *stabilizer lift* since it sufficiently stabilizes the subgrade to allow access by normal construction equipment.
5. Once the stabilizer lift is completed, construction proceeds using standard methods.

The design method assumes that the stabilizer lift is an unpaved road which will be exposed to relatively few vehicle passes (*i.e.*, construction equipment only) and which can tolerate 50 to 75 mm of rutting under the equipment loads. The design consists of the following steps:

STEP 1. Assess need for geotextile.

Estimate the need for a geotextile based on the subgrade strength and by past performance in similar types of soils.

STEP 2. Design pavement without geotextile.

Design the roadway for structural support using normal pavement design methods; provide no allowance for the geotextile.

STEP 3. Determine need for additional aggregate.

See Figure 5-7 to determine if additional aggregate above that required for structural support is needed due to susceptibility of soils to pumping and base course intrusion. If so, reduce that aggregate thickness and include a geotextile at the base/subgrade interface. Note that a thickness reduction of approximately 50% is normally cost effective.

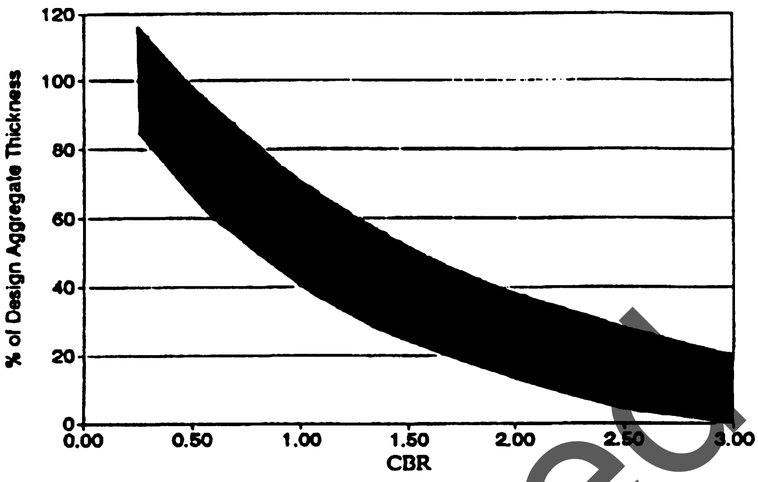


Figure 5-7 Aggregate loss to weak subgrades (FHWA, 1989; in Christopher and Holtz, 1991).

STEP 4. Determine aggregate depth required to support construction equipment.

Determine the additional aggregate required for stabilization of the subgrade during construction activities. Use a 50 to 75 mm rutting criteria for construction equipment, and refer to the procedures outlined in Section 5.6.

STEP 5. Compare thicknesses.

Compare the aggregate-geotextile system thicknesses determined in Steps 3 and 4. Use the system with the greater thickness.

STEP 6. Check geotextile filtration.

Check the geotextile filtration characteristics using the gradation and permeability of the subgrade, the water table conditions, and the retention and permeability criteria. From Chapter 2, that criteria is:

$AOS \leq D_{85}$ (Wovens) (Eq. 2-3)

$AOS \leq 1.8\ D_{85}$ (Nonwovens) (Eq. 2-4)

$k_{geotextile} \geq k_{soil}$ (Eq. 2-7a)

$\psi \geq 0.1\ sec^{-1}$ (Eq. 2-8c)

STEP 7. Determine geotextile survivability requirements.
Check the geotextile strength requirements for survivability as discussed in Section 5.5.

STEP 8. Specify geotextiles that meet or exceed those survivability criteria.

STEP 9. Follow the construction recommendations in Section 5.12.

Design methods for improving the structural capacity of permanent roads using geotextiles (*e.g.*, Hamilton and Pearce, 1981) and geogrids (*e.g.*, Haas, 1986; Haas, et. al., 1988; Barksdale, et al., 1989; Webster, 1993) have been proposed and may also be used. If a geogrid is used, either the base material should be sufficiently well graded to provide subgrade filtration and prevent soil intrusion, or for more open bases, a geotextile filter should be used with the geogrid.

5.9 PERMANENT ROAD DESIGN EXAMPLE (Christopher and Holtz, 1991)

DEFINITION OF DESIGN EXAMPLE

- Project Description: New public street and service drive for a suburban Washington, D.C., townhouse development. State of Virginia DOT regulations apply.
- Type of Structure: Category IV street (permanent road)
- Type of Application: geotextile separator
- Alternatives: i) excavate unsuitable material and increase subgrade aggregate thickness; or ii) geotextile separator between aggregate and subgrade

GIVEN DATA

- subgrade
  - surficial soils: micaceous silts (CBR ≈ 2)
  - local areas of very poor soils (CBR ≈ 0.5)
  - low-lying topography
  - poor drainage
  - other nearby streets and roads require frequent maintenance

- traffic
  - maximum 300 vehicles per day
  - 96% passenger, 5% single-axle, 1 multiaxle
  - equivalent daily 90 kN single-axle load (EAL) applications = 10

REQUIRED

Design the pavement section.

Consider: 1) standard AASHTO design; and 2) alternate with geotextile

DEFINE

A. Geotextile function(s):

B. Geotextile properties required:

C. Geotextile specification:

SOLUTION

A. Geotextile function(s):
Primary - separation
Secondary - filtration

B. Geotextile properties required:
survivability
apparent opening size (AOS)
permeability

DESIGN Design pavement with and without geotextile inclusion. Compare options.

STEP 1. ESTIMATE NEED FOR GEOTEXTILE

Ideal conditions for considering a geotextile; *e.g.*, low CBR, saturated subgrade, and poor performance history with conventional design.

STEP 2. DESIGN WITHOUT GEOTEXTILE

The structural design for the pavement section is based on the AASHTO Guide for Design of Pavement Structures (1977) using an equivalent design structural number for the anticipated loading and soil support conditions. (NOTES: i) AASHTO design uses English units; and ii) this case history used the AASHTO guide, 1977, which was current at time of design.)

traffic - as given

Determine structural number, SN:
from AASHTO design charts and with 20 years, CBR = 2, EAL = 10, and Regional Factor = 2
SN is equal to 2.9

Compute pavement thickness for structural support on a stable subgrade (*i.e.*, no fines pumped into aggregate subbase and no aggregate loss down into the subgrade):

Assume 2.5 inches asphaltic concrete surface and 8 inches aggregate base course

	surface	+	base	+	subbase
SN =	$a_1 D_1$	+	$a_2 D_2$	+	$a_3 D_3$
2.9 =	0.4 *x* 2.5"	+	0.14 *x* 8"	+	0.13 *x* D_3

Therefore, D_3 = 6 inches required for subbase.

Structural design: 2.5" asphaltic concrete
8" aggregate base
6" aggregate subbase

STEP 3. ADDITIONAL AGGREGATE FOR PUMPING AND INTRUSION

By local experience in this area, and for subgrades of CBR ≤ 2, an additional 8 inches of aggregate subbase is required (stabilization aggregate).

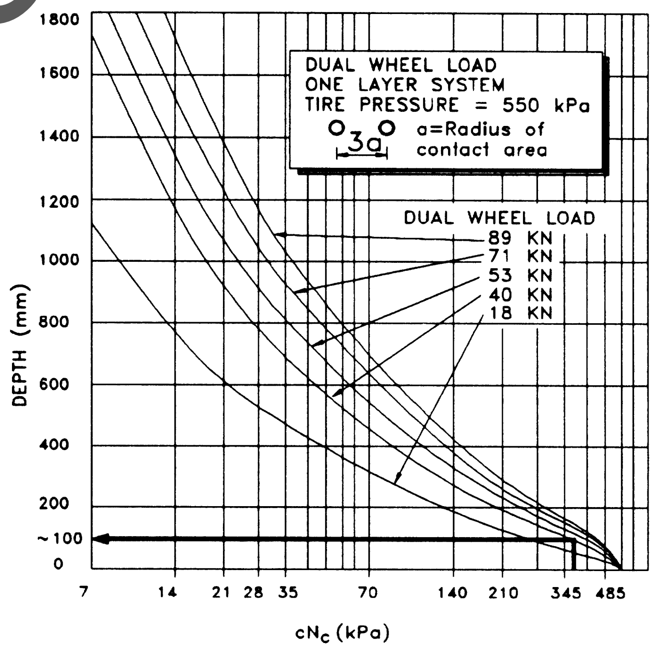
For the geotextile separator alternate, this entire stabilization layer could be eliminated. However, some very poor soils are anticipated, and some conservatism can be applied. Therefore, reduce subbase aggregate thickness to 4 inches (100 mm) with use of a geotextile separator.

STEP 4. DESIGN FOR CONSTRUCTABILITY USING A GEOTEXTILE

Use temporary road design procedures.

Assume:
CBR = 2
loaded dump trucks
< 100 passes
50 mm rut depth acceptable

Use Figure 5-5 (use 40 kN load) and
c ≈ 30 *x* CBR ≈ 60 kPa
N_c = 6
cN_c = 360 kPa
aggregate depth = 100 mm



STEP 5. COMPARE THE THICKNESSES DETERMINED IN STEPS 3 AND 4

The two thicknesses are equal; therefore, use 100 mm of stabilization aggregate with a geotextile separator. Note that the minimum thickness of aggregate for a construction haul road is 100 mm, though the contractor will likely use a greater thickness.

STEP 6. CHECK GEOTEXTILE FILTRATION CHARACTERISTICS

Use AOS < 0.3 mm per Table 5-1 because > 50% passing the 0.075 mm sieve.

Permeability of geotextile must be greater than soil permeability per Table 5-1. Estimate soil permeability and determine geotextile requirement.

STEP 7. DETERMINE GEOTEXTILE SURVIVABILITY REQUIREMENTS

Use Table 5-2, with CBR = 2, dump truck contact pressure > 350 kPa, and 150 mm cover thickness (note that 100 mm cover is limited to existing road bases, therefore 150 mm minimum compacted lift thickness is recommended), and determine that a geotextile with a **HIGH**, or Class 1, survivability rating is required.

STEP 8. SPECIFY GEOTEXTILE PROPERTY REQUIREMENTS

From Table 5-1; geotextile separator shall meet or exceed the minimum average roll values, with elongation at failure determined with the ASTM D 4632 test method, of:

Property	ASTM Test Method	Elongation $\leq$ 50%	Elongation $\geq$ 50%
Grab Strength	D 4632	1400	900
Sewn Seam Strength	D 4632	1200	810
Tear Resistance	D 4533	500	350
Puncture	D 4833	500	350
Burst	D 3786	3500	1700
Ultraviolet Stability	D 4355	50% strength retained after 500 hours	

The geotextile shall have an AOS < 0.3 mm, $\psi \geq 0.1$ sec^{-1}, and the permeability shall be ______.

STEP 9. SPECIFY CONSTRUCTION REQUIREMENTS

See Section 5.12

5.10 COST CONSIDERATIONS

Estimation of construction costs and benefit-cost ratios for geosynthetic-stabilized road construction is straight-forward and basically the same as that required for alternative pavement designs. Primary factors include the following:

1. cost of the geosynthetic;
2. cost of constructing the conventional design versus a geosynthetic design (*i.e.*, stabilization

requirements for conventional design versus geosynthetic design), including

a) stabilization aggregate requirements,
b) excavation and replacement requirements,
c) operational and technical feasibility, and
d) construction equipment and time requirements;

3. cost of conventional maintenance during pavement service life versus improved service anticipated by using geosynthetic (estimated through pavement management programs); and
4. regional experience.

Annual cost formulas, such as the Baldock method (Illinois DOT, 1982), can be applied with an appropriate present worth factor to obtain the present worth of future expenditures.

Cost tradeoffs should also be evaluated for different construction and geosynthetic combinations. This should include subgrade preparation and equipment control versus geosynthetic survivability. In general, higher-cost geosynthetics with a higher survivability on the existing subgrade will be less expensive than the additional subgrade preparation necessary to use lower-survivability geosynthetics.

Research is ongoing to quantify the cost-benefit life cycle ratio of using geosynthetics in permanent roadway systems. In any case, the cost of a geosynthetic is generally $1.25/m^2$ while the cost of the pavement section is generally $25/m^2$. **The life extension of the roadway section will more than make up for the cost of the geosynthetic. The ability of a geosynthetic to prevent premature failure provides an extremely low-cost performance insurance.**

5.11 SPECIFICATIONS

5.11-1 Geotextile for Separation and Stabilization Applications

Specifications should generally follow the guidelines in Section 1.6. The main considerations include the minimum geotextile requirements for design and those obtained from the survivability, retention, and filtration requirements in (Sections 5.5 and 5.8), as well as the construction requirements covered in Section 5.12. As with other applications, it is very important that an engineer's representative be on site during placement to observe that the correct geotextile has been delivered, that the specified construction sequence is being followed in detail, and that no damage to the geotextile is occurring. The following example specification is a combination of the AASHTO M288 (1997) geotextile material specification and its accompanying construction/installation guidelines.

SPECIFICATION FOR GEOTEXTILES USED IN SEPARATION AND STABILIZATION APPLICATIONS

(after AASHTO M288, 1997)

1. SCOPE

1.1 *Description.* This specification is applicable to the use of a geotextile to prevent mixing of a subgrade soil and an aggregate cover material (*i.e.*, separation application); and to the use of a geotextile in wet, saturated conditions to provide the coincident functions of separation and filtration (*i.e.*, stabilization application). In some stabilization applications, the geotextile can also provide the function of reinforcement.

1.2 *Separation.* The separation application is appropriate for pavement structures constructed over soils with a California Bearing Ratio greater than or equal to three (CBR $\geq$ 3) (shear strength greater than .pproximately 90 kPa). It is appropriate for unsaturated subgrade soils. The primary function of a geotextile in this application is separation.

1.3 *Stabilization.* The stabilization application is appropriate for subgrade soils which are saturated due to a high groundwater table or due to prolonged periods of wet weather. Stabilization is applicable to pavement structures constructed over soils with a CBR between one and three (1 < CBR < 3) (shear strength between approximately 30 kPa and 90 kPa). This specification is not appropriate for embankment reinforcement where stress conditions may cause global subgrade foundation or stability failure. Reinforcement of the pavement section is a site-specific design issue.

2. REFERENCED DOCUMENTS

2.1 *AASHTO Standards*

- T88 Particle Size Analysis of Soils
- T90 Determining the Plastic Limit and Plasticity Index of Soils
- T99 The Moisture-Density Relationships of Soils Using a 2.5 kg Rammer and a 305 mm Drop

2.2 *ASTM Standards*

- D 123 Standard Terminology Relating to Textiles
- D 276 Test Methods for Identification of Fibers in Textiles
- D 3786 Test Method for Hydraulic Burst Strength of Knitted Goods and Nonwoven Fabrics, Diaphragm Bursting Strength Tester Method
- D 4354 Practice for Sampling of Geosynthetics for Testing
- D 4355 Test Method for Deterioration of Geotextiles from Exposure to Ultraviolet Light and Water (Xenon Arc Type Apparatus)
- D 4439 Terminology for Geosynthetics
- D 4491 Test Methods for Water Permeability of Geotextiles by Permittivity
- D 4632 Test Method for Grab Breaking Load and Elongation of Geotextiles
- D 4751 Test Method for Determining Apparent Opening Size of a Geotextile
- D 4759 Practice for Determining the Specification Conformance of Geosynthetics
- D 4833 Test Method for Index Puncture Resistance of Geotextiles, Geomembranes and Related Products
- D 4873 Guide for Identification, Storage, and Handling of Geotextiles

D 5141 Test Method to Determine Filtering Efficiency and Flow Rate for Silt Fence Applications Using Site Specific Soil

3. PHYSICAL AND CHEMICAL REQUIREMENTS

3.1 Fibers used in the manufacture of geotextiles and the threads used in joining geotextiles by sewing, shall consist of long chain synthetic polymers, composed of at least 95% by weight polyolefins or polyesters. They shall be formed into a stable network such that the filaments or yarns retain their dimensional stability relative to each other, including selvages.

3.2 *Geotextile Requirements.* The geotextile shall meet the requirements of following Table. Woven slit film geotextiles (*i.e.*, geotextiles made from yarns of a flat, tape-like character) will not be allowed. All numeric values in the following table, except AOS, represent minimum average roll values (MARV) in the weakest principal direction (*i.e.*, average test results of any roll in a lot sampled for conformance or quality assurance testing shall meet or exceed the minimum values). Values for AOS represent maximum average roll values.

4. CERTIFICATION

4.1 The Contractor shall provide to the Engineer a certificate stating the name of the manufacturer, product name, style number, chemical composition of the filaments or yarns and other pertinent information to fully describe the geotextile.

4.2 The Manufacturer is responsible for establishing and maintaining a quality control program to assure compliance with the requirements of the specification. Documentation describing the quality control program shall be made available upon request.

4.3 The Manufacturer's certificate shall state that the furnished geotextile meets MARV requirements of the specification as evaluated under the Manufacturer's quality control program. The certificate shall be attested to be a person having legal authority to bind the Manufacturer.

4.4 Either mislabeling or misrepresentation of materials shall be reason to reject those geotextile products.

5. SAMPLING, TESTING, AND ACCEPTANCE

5.1 Geotextiles shall be subject to sampling and testing to verify conformance with this specification. Sampling for testing shall be in accordance with ASTM D 4354. Acceptance shall be based on testing of either conformance samples obtained using Procedure A of ASTM D 4354, or based on manufacturer's certifications and testing of quality assurance samples obtained using Procedure B of ASTM D 4354. A lot size for conformance or quality assurance sampling shall be considered to be the shipment quantity of the given product or a truckload of the given product, whichever is smaller.

5.2 Testing shall be performed in accordance with the methods referenced in this specification for the indicated application. The number of specimens to test per sample is specified by each test method. Geotextile product acceptance shall be based on ASTM D 4759. Product acceptance is determined by comparing the average test results of all specimens within a given sample to the specification MARV. Refer to ASTM D 4759 for more details regarding geotextile acceptance procedures.

Geotextile Requirements for Separation and Stabilization Applications

Property	ASTM Test Method	Units	Separation Application Class 2[1]		Stabilization Application Class 1[2]	
			Geotextile Elongation < 50%[3]	Geotextile Elongation $\geq$ 50%[3]	Geotextile Elongation < 50%[3]	Geotextile Elongation $\geq$ 50%[3]
Grab Strength	D 4632	N	1100	700	1400	900
Sewn Seam Strength[4]	D 4632	N	990	630	1200	810
Tear Strength	D 4533	N	400[5]	250	500	350
Puncture Strength	D 4833	N	400	250	500	350
Burst Strength	D 3786	kPa	2700	1300	3500	1700
Permittivity	D 4491	sec^{-1}	0.02[5]		0.05[5]	
Apparent Opening Size	D 4751	mm	0.60 max.		0.43 max.	
Ultraviolet Stability (Retained Strength)	D 4355	%	50% after 500 hours of exposure			

NOTES:

(1) Default geotextile selection. The Engineer may specify a Class 3 geotextile [Appendix D] based on one or more of the following:

  a) The Engineer has found Class 3 geotextiles to have sufficient survivability based on field experience.
  b) The Engineer has found Class 3 geotextiles to have sufficient survivability based on laboratory testing and visual inspection of a geotextile sample removed from a field test section constructed under anticipated field conditions.
  c) Aggregate cover thickness of the first lift over the geotextile exceeds 300 mm and aggregate diameter is less than 50 mm.
  d) Aggregate cover thickness of the first lift over the geotextile exceeds 150 mm, aggregate diameter is less than 30 mm, and construction equipment contact pressure is less than 550 kPa.

(2) Default geotextile selection. The Engineer may specify a Class 2 or 3 geotextile [Appendix D] based on one or more of the following:

  a) The Engineer has found the class of geotextile to have sufficient survivability based on field experience.
  b) The Engineer has found the class of geotextile to have sufficient survivability based on laboratory testing and visual inspection of a geotextile sample removed form a field test section constructed under anticipated field conditions.

(3) As measured in accordance with ASTM D 4632.

(4) When sewn seams are required.

(5) Default value. Permittivity of the geotextile should be greater than that of the soil ($\Psi_g > \Psi_s$). The Engineer may also require the permeability of the geotextile to be greater than that of the soil ($k_g > k_s$).

6. SHIPMENT AND STORAGE

6.1 Geotextile labeling, shipment, and storage shall follow ASTM D 4873. Product labels shall clearly show the manufacturer or supplier name, style number, and roll number. Each shipping document shall include a notation certifying that the material is in accordance with the manufacturer's certificate.

6.2 Each geotextile roll shall be wrapped with a material that will protect the geotextile from damage due to shipment, water, sunlight, and contaminants. The protective wrapping shall be maintained during periods of shipment and storage.

6.3 During storage, geotextile rolls shall be elevated off the ground and adequately covered to protect them from the following: site construction damage, precipitation, extended ultraviolet radiation including sunlight, chemicals that are strong acids or strong bases, flames including welding sparks, temperatures in excess of 71°C (160°F), and any other environmental condition that may damage the physical property values of the geotextile.

7. CONSTRUCTION

7.1 *General.* Atmospheric exposure of geotextiles to the elements following lay down shall be a maximum of 14 days to minimize damage potential.

7.2 *Seaming.*

a. If a sewn seam is to be used for the seaming of the geotextile, the thread used shall consist of high strength polypropylene, or polyester. Nylon thread shall not be used. For erosion control applications, the thread shall also be resistant to ultraviolet radiation. The thread shall be of contrasting color to that of the geotextile itself.

b. For seams which are sewn in the field, the Contractor shall provide at least a 2 m length of sewn seam for sampling by the Engineer before the geotextile is installed. For seams which are sewn in the factory, the Engineer shall obtain samples of the factory seams at random from any roll of geotextile which is to be used on the project.

b.1 For seams that are field sewn, the seams sewn for sampling shall be sewn using the same equipment and procedures as will be used for the production of seams. If seams are to be sewn in both the machine and cross machine directions, samples of seams from both directions shall be provided.

b.2 The seam assembly description shall be submitted by the Contractor along with the sample of the seam. The description shall include the seam type, stitch type, sewing thread, and stitch density.

7.3 *Site Preparation.* The installation site shall be prepared by clearing, grubbing, and excavation or filling the area to the design grade. This includes removal of top soil and vegetation.

NOTE: Soft spots and unsuitable areas will be identified during site preparation or subsequent proof rolling. These areas shall be excavated and backfilled with select material and compacted using normal procedures.

7.4 *Geotextile Placement.*

a. The geotextile shall be laid smooth without wrinkles or folds on the prepared subgrade in the direction of construction traffic. Adjacent geotextile rolls shall be overlapped, sewn or joined as required in the plans. Overlaps shall be in the direction as shown on the plans. See following Table for overlap requirements.

Overlap Requirements

SOIL CBR	MINIMUM OVERLAP
Greater than 3	300 - 450 mm
1 - 3	0.6 - 1 m
0.5 - 1	1 m or sewn
Less than 0.5	Sewn
All Roll Ends	1 m or sewn

a.1 On curves the geotextile may be folded or cut to conform to the curves. The fold or overlap shall be in the direction of construction and held in place by pins, staples, or piles of fill or rock.

a.2 Prior to covering, the geotextile shall be inspected by a certified inspector of the Engineer to ensure that the geotextile has not been damaged (*i.e.*, holes, tears, rips) during installation. Damaged geotextiles, as identified by the Engineer, shall be repaired immediately. Cover the damaged area with a geotextile patch which extends an amount equal to the required overlap beyond the damaged area.

b. The subbase shall be placed by end dumping onto the geotextile from the edge of the geotextile, or over previously placed subbase aggregate. Construction vehicles shall not be allowed directly on the geotextile. The subbase shall be placed such that at least the minimum specified lift thickness shall be between the geotextile and equipment tires or tracks at all times. Turning of vehicles shall not be permitted on the first lift above the geotextile.

NOTE: On subgrades having a CBR values of less than 1, the subbase aggregate should be spread in its full thickness as soon as possible after dumping to minimize the potential of localized subgrade failure due to overloading of the subgrade.

b.1 Any ruts occurring during construction shall be filled with additional subbase material, and compacted to the specified density.

b.2 If placement of the backfill material causes damage to the geotextile, the damaged area shall be repaired as previously described in section 7.4.a.2. The placement procedures shall then be modified to eliminate further damage from taking place. (*i.e.*, increased initial lift thickness, decrease equipment loads, etc.)

NOTE: In stabilization applications, the use of vibratory compaction equipment is not recommended with the initial lift of subbase material, as it may cause damage to the geotextile.

8. METHOD OF MEASUREMENT

8.1 The geotextile shall be measured by the number of square meters computed from the payment lines shown on the plans or from payment lines established in writing by the Engineer. This excludes seam overlaps, but shall include geotextiles used in crest and toe of slope treatments.

8.2 Slope preparation, excavation and backfill, bedding, and cover material are separate pay items.

9. BASIS OF PAYMENT

9.1 The accepted quantities of geotextile shall be paid for per square meter in place.

9.2 Payment will be made under:

Pay Item	Pay Unit
Separation Geotextile	Square Meter
Stabilization Geotextile	Square Meter

5.11-2 Geogrid Reinforcement

An AASHTO, or other standard setting organization, geogrid specification for reinforcement of pavement structures is, presently, not available. Nor was a widely accepted, typical state agency specification for geogrid reinforcement located for inclusion in this manual (though several agency's do have a geogrid reinforcement specification). A typical, generic type material specification for geogrid reinforcement for pavements will be difficult to develop because of: the proprietary nature (i.e., current product patents) of biaxial geogrids; the absence of generic design procedure; a lack of understanding of the mechanistic benefits of geogrid reinforcement; lack of a clear definition of the function(s) of the geogrid in pavement reinforcement application; lack of side-by-side product performance testing; lack of performance documentation; and inability to measure contribution of geogrid reinforcement to pavement structure with non-destructive testing methods.

Agencies which are using geogrids in pavements have typically initiated use after the following considerations:

1. Define purpose and applicability geogrid reinforcement. For example, geogrid reinforcement may be used to minimize over excavation over soft subgrades, to reduce or minimize aggregate base course thickness, to extend pavement life (i.e., analysis period), or a combination of these.
2. Define function(s) of geosynthetic, and assess applicability of geogrids.
3. Construct and monitor a demonstration project to examine performance of candidate geogrid reinforcement(s) versus the agency's conventional construction technique.
4. With satisfactory performance, write a material specification which either lists prequalified products or lists key property requirements.
5. Monitoring of additional projects to confirm anticipated performance, and construction of additional demonstration projects to confirm performance of new products, as needed.

Thus, a geogrid pavement reinforcement specification may list prequalified products or key property requirements. Key property requirements may include definition of some, or all, of the following properties: aperture size; percent open area; tensile modulus (initial, or 2% or 5% secant); rib junction strength; rib junction efficiency; rib thickness; flexural rigidity; and secant aperture stability. See Webster (1993) for a description of the aperture stability test procedure, and applicability of results to their test program.

A geogrid reinforcement specification should allow use of other geogrid products that either: (i) meet the physical properties defined by the key property requirements; or (ii) by demonstrating performance equivalency through full-scale laboratory testing, in-ground testing of pavements, and prior projects.

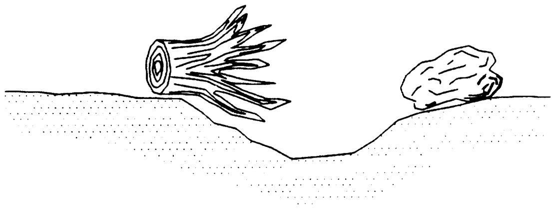
5.12 INSTALLATION PROCEDURES

5.12-1 Roll Placement

Successful use of geotextiles in pavements requires proper installation, and Figure 5-8 shows the proper sequence of construction. Even though the installation techniques appear fairly simple, most geotextile problems in roadways occur as the result of improper construction techniques.

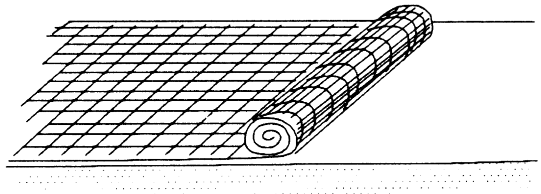
If the geotextile is ripped or punctured during construction activities, it will not likely perform as desired. If the geotextile is placed with a lot of wrinkles or folds, it will not be in tension, and, therefore, cannot provide a reinforcing effect. Other problems occur due to insufficient cover over the geotextile, rutting of the subgrade prior to placing the geotextile, and thin lifts that exceed the bearing capacity of the soil. The following step-by-step procedures should be followed, along with careful observations of all construction activities.

1. The site should be cleared, grubbed, and excavated to design grade, stripping all topsoil, soft soils, or any other unsuitable materials (Figure 5-8a). If moderate site conditions exist, *i.e.*, CBR greater than 1, lightweight proofrolling operations should be considered to help locate unsuitable materials. Isolated pockets where additional excavation is required should be backfilled to promote positive drainage. Optionally, geotextile-wrapped trench drains could be used to drain isolated areas.
2. During stripping operations, care should be taken not to excessively disturb the subgrade. This may require the use of lightweight dozers or grade-alls for low-strength, saturated, noncohesive and low-cohesive soils. For extremely soft ground, such as peat bog areas, do not excavate surface materials so you may take advantage of the root mat strength, if it exists. In this case, all vegetation should be cut at the ground surface. Sawdust or sand can be placed over stumps or roots that extend above the ground surface to cushion the



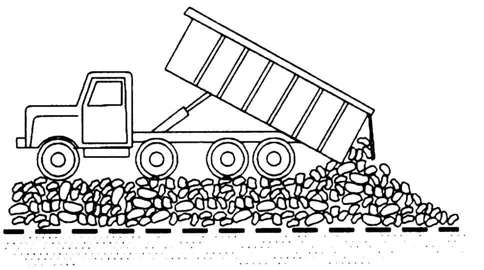
a. Prepare the ground by removing stumps, boulders, etc.; fill in low spots

PREPARE THE GROUND



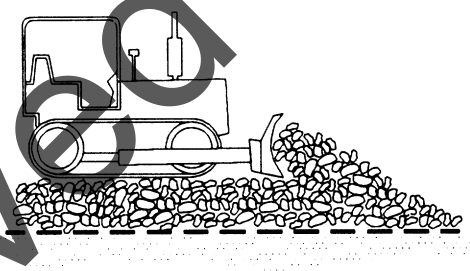
b. Unroll the geotextile directly over the ground to be stabilized. If more than one roll is required, overlap rolls. Inspect geotextile.

UNROLL THE GEOTEXTILE



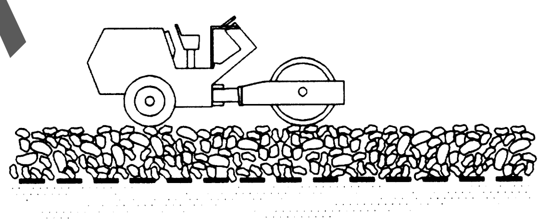
c. Back dump aggregate onto previously placed aggregate. Do not drive on the geotextile. Maintain 150 mm to 300 mm cover between truck tires and Geotextile.

BACK DUMP AGGREGATE



d. Spread the aggregate over the geotextile to the design thickness.

SPREAD THE AGGREGATE



e. Compact the aggregate using dozer tracks or smooth drum vibratory roller.

COMPACT THE AGGREGATE

Figure 5-8 Construction sequence using geotextiles.

geotextile. Remember, the subgrade preparation must correspond to the survivability properties of the geotextile.

3. Once the subgrade along a particular segment of the road alignment has been prepared, the geotextile should be rolled in line with the placement of the new roadway aggregate (Figure 5-8b). Field operations can be expedited if the geotextile is pre-sewn to design widths in the factory so it can be unrolled in one continuous sheet. The geotextile should not be dragged across the subgrade. The entire roll should be placed and rolled out as smoothly as possible. Wrinkles and folds in the fabric should be removed by stretching and staking as required.
4. Parallel rolls of geotextiles should be overlapped, sewn, or joined as required. (Specific requirements are given in Sections 5.12-2 and 5.12-3.)
5. For curves, the geotextile should be folded or cut and overlapped in the direction of the turn (previous fabric on top) (Figure 5-9). Folds in the geotextile should be stapled or pinned approximately 0.6 m on centers.
6. When the geotextile intersects an existing pavement area, the geotextile should extend to the edge of the old system. For widening or intersecting existing roads where geotextiles have been used, consider anchoring the geotextile at the roadway edge. Ideally, the edge of the roadway should be excavated down to the existing geotextile and the existing geotextile sewn to the new geotextile. Overlaps, staples, and pins could also be utilized.
7. Before covering, the condition of the geotextile should be checked for excessive damage (*i.e.*, holes, rips, tears, etc.) by an inspector experienced in the use of these materials. If excessive defects are observed, the section of the geotextile containing the defect should be repaired by placing a new layer of geotextile over the damaged area. The minimum required overlap required for parallel rolls should extend beyond the defect in all directions. Alternatively, the defective section can be replaced.
8. The base aggregate should be end-dumped on the previously placed aggregate (Figure 5-8c). For very soft subgrades, pile heights should be limited to prevent possible subgrade failure. The maximum placement lift thickness for such soils should not exceed the design thickness of the road.
9. The first lift of aggregate should be spread and graded to 300 mm, or to the design thickness if less than 300 mm, prior to compaction (Figure 5-8d). At no time should traffic be allowed on a soft roadway with less than 200 mm (150 mm for CBR $\geq$ 3) of aggregate over the geotextile. Equipment can operate on the roadway without aggregate for geotextile installation under permeable bases, if the subgrade is of sufficient strength. For extremely soft soils, lightweight construction vehicles will likely be required for access on the first lift. Construction vehicles should be limited in size and weight so rutting in the initial lift is limited to 75 mm. If rut depths exceed 75 mm, it will be necessary to decrease the construction vehicle size and/or weight or to increase the lift thickness. For

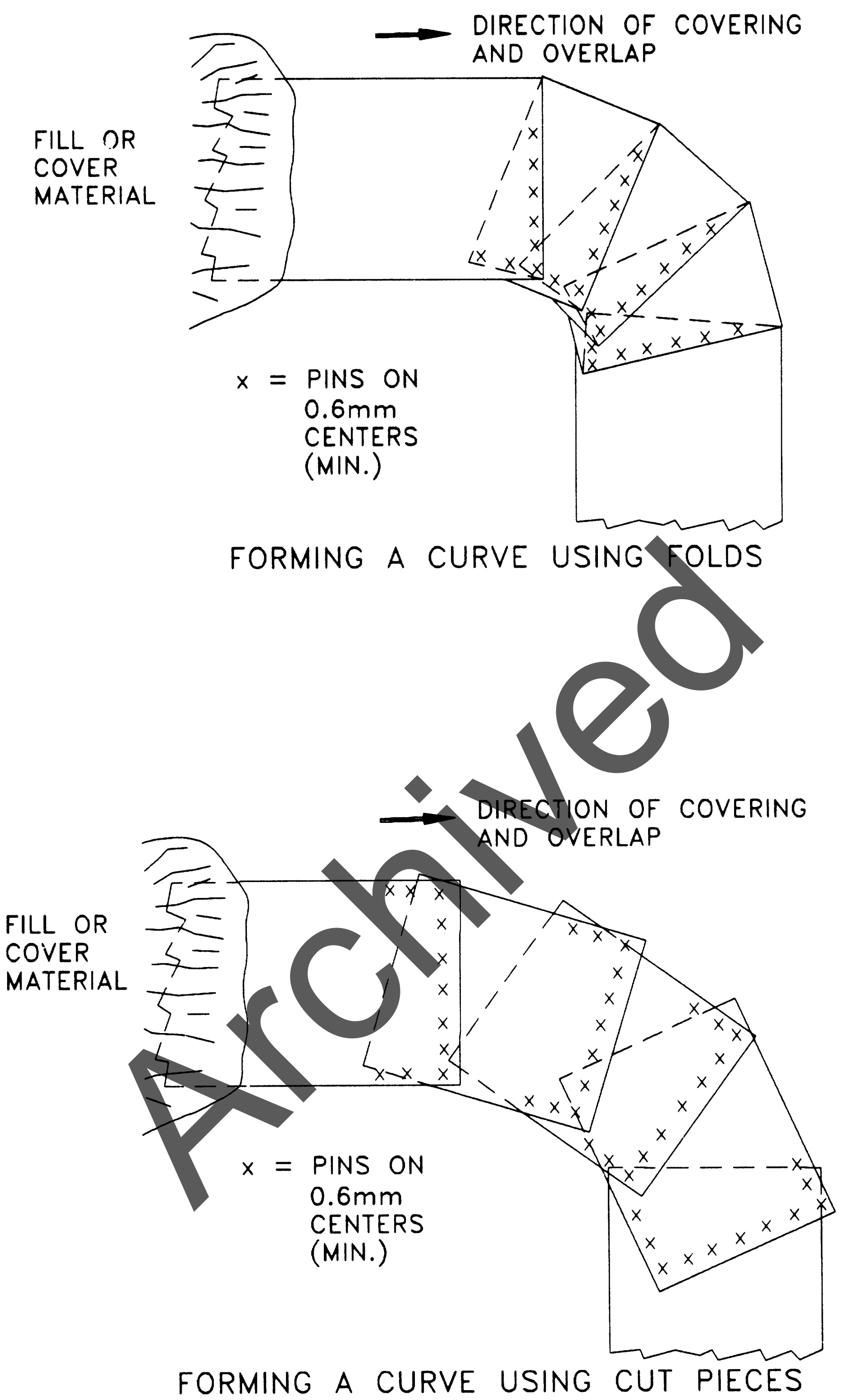


Figure 5-9 Forming curves using geotextiles.

example, it may be necessary to reduce the size of the dozer required to blade out the fill or to deliver the fill in half-loaded rather than fully loaded trucks.

10. The first lift of base aggregate should be compacted by *tracking* with the dozer, then compacted with a smooth-drum vibratory roller to obtain a minimum compacted density (Figure 5-8e). For construction of permeable bases, compaction shall meet specification requirements. For very soft soils, design density should not be anticipated for the first lift and, in this case, compaction requirements should be reduced. One recommendation is to allow compaction of 5% less than the required minimum specification density for the first lift.
11. Construction should be performed parallel to the road alignment. Turning should not be permitted on the first lift of base aggregate. Turn-outs may be constructed at the roadway edge to facilitate construction.
12. On very soft subgrades, if the geotextile is to provide some reinforcing, pretensioning of the geotextile should be considered. For pretensioning, the area should be proofrolled by a heavily loaded, rubber-tired vehicle such as a loaded dump truck. The wheel load should be equivalent to the maximum expected for the site. The vehicle should make at least four passes over the first lift in each area of the site. Alternatively, once the design aggregate has been placed, the roadway could be used for a time prior to paving to prestress the geotextile-aggregate system in key areas.
13. Any ruts that form during construction should be filled in, as shown in Figure 5-10 to maintain adequate cover over the geotextile. In no case should ruts be bladed down, as this would decrease the amount of aggregate cover between the ruts.
14. All remaining base aggregate should be placed in lifts not exceeding 250 mm in loose thickness and compacted to the appropriate specification density.

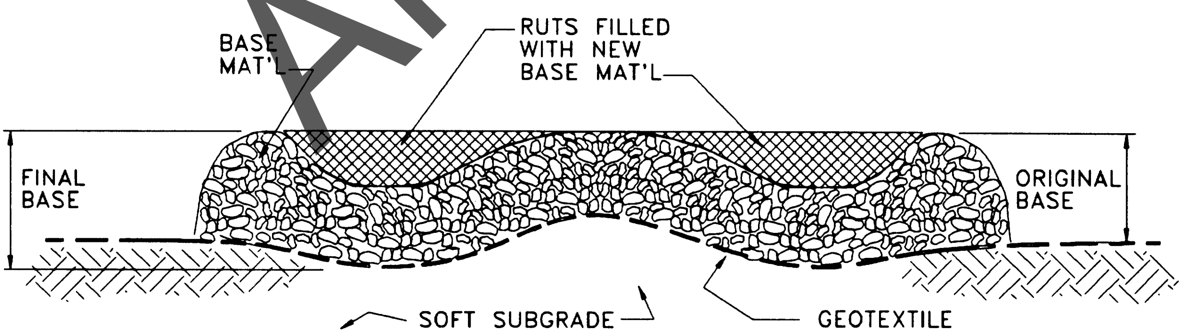


Figure 5-10 Repair of rutting with additional material.

5.12-2 Geotextile Overlaps

Overlaps can be used to provide continuity between adjacent geotextile rolls through frictional resistance between the overlaps. Also, a sufficient overlap is required to prevent soil from squeezing into the aggregate at the joint. The amount of overlap depends primarily on the soil conditions and the potential for equipment to rut the soil. If the subgrade does not rut under construction activities, only a minimum overlap is required to provide some pullout resistance. As the potential for rutting and squeezing of soil increases, the required overlap increases. Since rutting potential can be related to CBR, it can be used as a guideline for the minimum overlap required, as shown in Table 5-4.

TABLE 5-4
RECOMMENDED MINIMUM GEOTEXTILE OVERLAP REQUIREMENTS

CBR	Minimum Overlap
> 2	300 - 450 mm
1 - 2	600 - 900 mm
0.5 - 1	900 mm or sewn[1]
< 0.5	sewn[1]
All roll ends	900 mm or sewn[1]
NOTE: 1. See Section 5.12-3.	

The geotextile can be stapled or pinned at the overlaps to maintain their position during construction activities. Nails 250 to 300 mm long should be placed at a minimum of 15 m on centers for parallel rolls and 1.5 m on centers for roll ends.

Geotextile roll widths should be selected so overlaps of parallel rolls occur at the roadway centerline and at the shoulders. Overlaps should not be placed along anticipated primary wheel path locations. Overlaps at the end of rolls should be in the direction of the aggregate placement (previous roll on top).

5.12-3 Seams

When seams are required for separation applications, they should meet the same tensile strength requirements for survivability (Table 5-1) as those of the geotextile perpendicular to the seam (as determined by the same testing methods). Seaming is discussed in detail in Section 1.8. All factory or field seams should be sewn with thread as strong and durable as the material in the fabric. *J-seams* with interlocking stitches are recommended. Alternatively, if bag-type stitches, which can easily unravel, or butt-type seams are used, seams should be double-sewn with parallel stitching spaced no more than 5 to 10 mm apart. Double sewing is required to safeguard against undetected missed stitches. The geotextile strength may actually have to exceed the specifications in order to provide seam strengths equal to the specified tensile strength.

For certain geogrids, overlap joints, tying or interlocking with wire cables, plastic pipe, hog rings, or bodkin joints may be required. Geotextile seam strength requirements should also be applied to overlapped or mechanically fastened geogrids. Consult the manufacturer for specific recommendations and strength test data.

5.13 FIELD INSPECTION

The field inspector should review the field inspection guidelines in Section 1.7. Particular attention should be paid to factors that affect geotextile survivability: subgrade condition, aggregate placement, lift thickness, and equipment operations.

5.14 SELECTION CONSIDERATIONS

For a geotextile to perform its intended function as a separator in a roadway, it must be able to tolerate the stresses imposed on it during construction; *i.e.*, the geotextile must have sufficient survivability to tolerate the anticipated construction operations. Geotextile selection for roadways is usually controlled by survivability, and the guidelines given in Section 5.5 are important in this regard. As mentioned, the specific geotextile property values given in Table 5-2 are minimums. For important projects, you are strongly encouraged to conduct your own field trials, as described in Section 5.5.

5.15 REFERENCES

References quoted within this section are listed below. Additional discussion on recent roadway research is presented in Appendix G. Detailed lists of specific ASTM and GRI test procedures are presented in Appendix E. The Koerner (1994) is a recent, comprehensive textbook on geosynthetics and is a *key* reference for design. This and other *key* references are noted in bold type.

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6.0 PAVEMENT OVERLAYS

6.1 BACKGROUND

The second largest application of geotextiles in North America is in asphalt overlays of asphalt concrete (AC) and Portland cement concrete (PCC) pavement structures. (The largest single application is separation/stabilization, which utilizes an estimated 100 million square meters of geotextile.) An estimated 85 million square meters of geotextiles were used in overlays in 1993 (GFR, 1993). This is 26% of the estimated 330 million square meters used in North America in 1993. This is indeed an impressive statistic. Notwithstanding, use of geotextiles in overlays may be described as a love-hate relationship with user agencies.

Many engineers are thoroughly convinced of the performance and cost benefit of geotextiles incorporated into overlays. Many other engineers are thoroughly convinced that geotextiles are either not beneficial or not economical in overlay construction. And, still other engineers are confused by the claims of performance and cost benefits. These divergent opinions regarding performance and benefits are addressed in this chapter.

The history of geotextiles in this application accounts for much of the confusion and skepticism. Promotion of geotextiles in overlays in the 1970s and early 1980s claimed that the geotextile *reinforced* the pavement and that the reinforcement *prevented* cracks in the old pavement from *reflecting* up through the new overlay. These claims are rarely, if ever, presented today. The tensile moduli of the light-weight nonwoven geotextiles typically used in this application are too low to mobilize significant tension under acceptable pavement deflections for the geotextile to act as a reinforcement. It is also commonly accepted that geotextiles do not *prevent* reflection cracking from occurring. How, then, are geotextiles beneficial in overlay pavement construction?

6.2 GEOTEXTILE FUNCTIONS

Geotextiles can be used as alternatives to stress-relieving granular layers, seal coats, rubberized asphalts, etc., for controlling surface moisture infiltration and retarding reflection cracks in pavement overlays. Properly installed, asphalt-saturated geotextiles function as a moisture barrier that protects the underlying pavement structure from further degradation due to ingress of surface water. In addition, geotextiles can provide cushioning for the overlay, thus functioning as a stress-relieving interlayer. When properly installed, both functions combine to extend the life of the overlay and the pavement section.

6.3 APPLICATIONS

Pavement rehabilitation is required where structural deficiencies adversely affect the load-carrying capability of the existing pavement structure. Impregnated geotextiles are used beneath asphalt concrete (AC) overlays for rehabilitation and as preventive measures to slow the deterioration rate of existing roadways. Preventive measures such as AC overlays and moisture protection are used to extend pavement life and are often economical when life-cycle costs are computed.

6.3-1 Asphalt Concrete Pavements

When geotextiles are used with AC overlays of AC pavements, they can be effective in controlling (retarding) reflection cracking of low- and medium-severity alligator-cracked pavements. They also may be useful for controlling reflection of thermal cracks, although they are not as effective in retarding reflection of cracks due to significant horizontal or vertical movements. (AASHTO, 1993)

The variable performance of geotextile overlays, and the divergent opinions regarding benefits, is strongly influenced by the following factors (Barksdale, 1991):

- type and extent of existing pavement distress, including crack widths;
- extent of remedial work performed on the old pavement, such as crack sealing and/or filling, pothole repair, and replacement of failed base and subgrade areas;
- overlay thickness;
- variability of pavement structural strength from one section to another; and
- climate.

Obviously, an additional factor is the geotextile. These factors are summarized below.

Distress Type (Barksdale, 1991): Geotextiles generally have performed best when used for load-related fatigue distress (*e.g.*, closely spaced alligator cracking). Fatigue cracks should be less than 3 mm wide for best results. Cracks greater than 10 mm wide require a stiff filler. Geotextiles used to retard thermal cracking have, in general, been found to be ineffective.

Remediation of Old Pavement (AASHTO, 1993): Much of the deterioration that occurs in overlays is the result of unrepaired distress in the existing pavement prior to the overlay. Distressed areas of the existing pavement should be repaired if the distress is likely to affect performance of the overlay within a few years. The amount of preoverlay repair is related to the overlay design. The engineer should consider the cost implications of preoverlay repair versus overlay design. Guidelines on preoverlay repair techniques are available from the FHWA (FHWA, {current version}; FHWA, 1987).

Effect of Overlay Thickness: Pavement performance is quite sensitive to the overlay thickness, either with or without a geotextile interlayer. Correspondingly, the benefits of a geotextile in retarding reflective cracking will increase with increasing thickness of the overlay. Geotextiles are most effective in retarding reflective cracking in thin (*e.g.*, 40 mm) overlays. These observations, as reported by Barksdale (1991), are based upon extensive research conducted by Caltrans (Predoehl, 1990). The California results imply that a relatively thin geotextile interlayer is structurally adequate and equivalent to 30 mm of asphalt concrete.

Variability of Pavement Structural Strength (Barksdale, 1991)**:** The structural strength of existing pavement, and, therefore, required overlay thickness, often varies greatly along a roadway. The significant effect of such variation on overlay performance has not often been considered in the past. This oversight likely contributes to some of the diverse opinions regarding geotextile benefits in asphalt overlays. (Variation of pavement strength along a roadway should be addressed for future demonstration or test sections of overlays.)

Climate: It has been observed (Aldrich, 1986) geotextile interlayers have generally performed better in warm and mild climates than in cold climates. However, the beneficial effects of reducing water infiltration - a principal function of the geotextile - were not considered in Aldrich's (1986) study. Successful installation and beneficial performance of geotextile interlayers in cold regions, such as Alaska, challenge the generality regarding climate.

Geotextile: Lightweight (*e.g.*, 135 g/m^2) nonwoven geotextiles are typically used for asphalt overlays. These asphalt-impregnated geotextiles primarily function as a moisture barrier. Use of heavier, nonwoven geotextiles can provide cushioning or stress-relieving membrane interlayer-like benefits, in addition to moisture-barrier functions.

6.3-2 Portland Cement Concrete Pavements

Geotextiles are used with AC overlays of crack/seat-fractured plain Portland cement concrete (PCC) pavement to help control reflection cracking (AASHTO, 1993).

Geotextiles also may be used for AC overlays of jointed plain concrete pavement (JPCP) and of jointed reinforced concrete pavement (JRCP) to control reflective cracking. However, the effectiveness with these pavements is listed as questionable in the AASHTO Guide for Design of Pavement Structures (1993).

Important factors in assessing applicability and potential benefits of using a geotextile interlayer with PCC pavements include:

- existing structural strength of the pavement;
- slab preparation;

- geotextile installation;
- required overlay thickness;
- climate; and
- economics of geotextile overlay versus other design alternatives (Barksdale, 1991).

6.3-3 AC-Overlaid PCC Pavements

Geotextiles are also used for new AC overlays of AC-overlaid Portland cement concrete pavements (AC/PCC), where the original pavements may be JPCP, JRCP or continuously reinforced concrete pavement (CRCP). Some pavements are constructed as AC/PCC, although most are PCC pavements that have already been overlaid with AC. In addition to controlling reflecting cracking, an impregnated geotextile can help control surface water infiltration into the pavement, which can result in loss of bond between AC and PCC, stripping in the AC layers, progression of *D* cracking or reactive aggregate distress (in pavements with these problems), and weakening of the base and subgrade materials.

6.4 ADVANTAGES AND POTENTIAL DISADVANTAGES

6.4-1 Advantages

An asphalt-impregnated geotextile functioning as a moisture barrier and a stress-relieving interlayer provides several possible benefits and, therefore, advantages to their use. Retardation of reflection cracks will:

- increase the overlay and the roadway life;
- decrease roadway maintenance costs; and
- increase pavement serviceability.

Reflection cracking can have a considerable, often controlling, influence on the life of an AC overlay (AASHTO, 1993). After a pavement cracks, its longevity is quickly reduced. Deteriorated reflection cracks require more frequent maintenance, such as sealing and patching. Reflection cracks also permit water to enter the pavement structure, which can weaken the base layers and subgrade, and decrease the structural capacity of the pavement. Base and subgrade will be weakened by ingress of water if the base does not have excellent (*i.e.*, water removed within 2 hours) or good (*i.e.*, water removed within 1 day) drainage. Water infiltration causes a reduction in shear strength and the subgrade, which in turn leads to a rapid deterioration of the pavement system. The sealing function of the asphalt-impregnated geotextile is intended to reduce surface water infiltration through reflection cracks (when they eventually reappear at the surface of the overlay) and through thermal-induced cracks.

Reduction in surface water entering PCC pavements potentially provides additional benefits of:

- reduction or elimination of pumping (*i.e.*, no water, no pumping);
- decreased slab movements through reduced erosion of fines from beneath the slab (lower moisture gradients might also reduce slab warping); and
- increased subgrade strength through a decrease in moisture (Barksdale, 1991).

6.4-2 Potential Disadvantages

Correct construction of the asphalt-impregnated geotextile and AC overlay is paramount to its functioning as designed. Too little asphalt in the tack coat can result in a partially saturated geotextile, which in turn can absorb moisture and lead to spalling or popping off of the surface treatment due to freeze-thaw action within the geotextile. Bleeding occurs with too much asphalt which can result in overlay slippage, as well as potential pavement slippage planes. Bleeding also can cause difficulty with installation, as it can result in the geotextile sticking to and being pulled up by the tires and tracks of the asphalt trucks and paving vehicle. The AC overlay must be placed below the specified temperature, which requires inspection and control. AC placement significantly above the specified temperature can result in the asphalt tack coat being drawn out of the geotextile which can result in shrinkage or even melting of the geotextile. Shrinkage and melting is a concern for a polypropylene geotextile which has a typical melt temperature of 165 °C, it is not a concern for a polyester geotextile which has a typical melt temperature of 225 °C. Improper pavement preparation and crack filling can also decrease the effectiveness of the geotextile moisture barrier.

Geotextiles cannot be expected to perform well when the roadway being overlaid is structurally inadequate. Nor will such surface treatments do anything to solve ground water problems, subgrade softening, base course contamination, or freeze-thaw problems. These problems must be corrected before resurfacing, independent of geotextile used.

Geotextiles have also been found ineffective in reducing thermal cracking. Pavement overlay systems have also had limited success in areas of heavy rainfall and regions with significant freeze-thaw (FHWA Manual, 1982).

6.5 DESIGN

6.5-1 General

Design of AC overlays is thoroughly presented in the AASHTO Guide for Design of Pavement Structures (1993). To have a high probability of success, a carefully planned and executed study is required to develop an engineered overlay design using a geotextile (Barksdale, 1991). A carefully planned and executed study also is required for successful, alternative (*i.e.*, non-geotextile) overlay designs.

The steps required to develop an overlay design for flexible pavements with a geotextile, as summarized from Barksdale (1991), are as follows.

STEP 1. Pavement condition evaluation.

The results of a general pavement condition survey are valuable in establishing the type, severity, and extent of pavement distress. Candidate pavements should be divided into segments, and a thorough visual evaluation made of each segment to determine the type, extent, and severity of cracking, and to classify the present distress as: alligator cracking, block cracking, transverse cracks, joint cracking, patching, potholes, widening drop-offs, etc. Crack widths should be measured. See AASHTO Guide for Design of Pavement Structures (1993) for guidance.

STEP 2. Structural strength.

The overall structural strength of the pavement should be evaluated along its length, using suitable nondestructive techniques, such as the Benkelman beam, falling-weight deflectometer, Dynaflect, or Road Rater.

STEP 3. Base/subgrade failure.

Areas with base or subgrade failures should be identified. Benkelman beam pavement deflections greater than approximately 0.6 mm are indicative of failure, as is excessive rutting.

STEP 4. Remedial pavement treatment.

The results of the pavement condition survey and deflection measurements should be used to develop a pavement repair strategy for each segment.

STEP 5. Overlay design.

A realistic overlay thickness must be selected to ensure a reasonable overlay life. Design methodologies are presented in the AASHTO Guide for Design of Pavement Structures (1993). The overlay thickness with a geotextile should be determined as if the interlayer is not present.

STEP 6. Geotextile selection.

STEP 7. Performance monitoring.

Performance monitoring during the life of the overlay is highly desirable for developing a local data bank of performance histories using geotextiles in overlays. Using a control section without a geotextile interlayer, with all other items equal, will yield valuable comparative data.

The steps in developing an overlay design for PCC pavements where a geotextile may be used is generally similar to that for flexible pavements (Barksdale, 1991). Vertical joint deflection surveys should be performed. Full-width geotextiles should not be used when vertical joint deflections are greater than about 0.2 mm, unless corrective measures such as undersealing, are taken to reduce joint movement. Horizontal thermal joint movement should be less than about 1.3 mm. As before, the thickness of the overlay is not reduced with the use of a geotextile interlayer.

6.5-2 Drainage Considerations

As noted previously, the primary function of the geotextile in an overlay is to minimize infiltration of surface water into the pavement structure. The benefits of this are normally not quantified and, if incorporated into design, are only subjectively treated. To objectively quantify the benefits of a moisture barrier a potential design approach is to estimate the effects of the asphalt-impregnated geotextile barrier on the drainage characteristics of the pavement structure. The 1993 AASHTO Guide for Design of Pavement Structures presents provisions for modifying pavement design equations to take advantage of performance improvements due to good drainage. Although not discussed in the design guide, a geotextile overlay could be considered a method to improve drainage via reduced infiltration.

From the AASHTO Guide (1993), modified layer coefficients determine the treatment for the expected drainage level for flexible pavements. The factor for modifying the layer coefficient is referred to as an m_i value, thus the structural number (SN) becomes:

$$SN = a_1 D_1 + a_2 D_2 m_2 + a_3 D_3 m_3$$

where:

D_1, D_2, D_3 = thicknesses of existing pavement surface, base, and subbase layers;
a_1, a_2, a_3 = corresponding structural layer coefficients; and
m_2, m_3 = drainage coefficients for granular base and subbase.

The recommended m_i values are presented in Table 6-1 as a function of the drainage quality and the percent of time during the year the pavement structure is near saturation. Definitions of quality of drainage are presented in Table 6-2.

From the AASHTO guide (1993), the drainage coefficient, C_d, in the performance equation determines the treatment for the expected drainage level for rigid pavements. The performance equation is used to calculate a design slab thickness for a rigid pavement. Recommended values for C_d are presented in Table 6-3.

Again, these modification factors are not discussed for use with geotextile overlays in the AASHTO design guide. They are presented here, as it is hypothesized that these values could aid in objectively estimating the structural benefit of a geotextile moisture barrier.

TABLE 6-1
RECOMMENDED m_i VALUES FOR MODIFYING STRUCTURAL LAYER COEFFICIENTS OF UNTREATED BASE AND SUBBASE MATERIALS IN FLEXIBLE PAVEMENTS
(from AASHTO, 1993)

Quality of Drainage	Percent of Time Pavement Structure is Exposed to Moisture Levels Approaching Saturation			
	< 1 %	1 to 5 %	5 to 25 %	> 25 %
Excellent	1.40 - 1.35	1.35 - 1.30	1.30 - 1.20	1.20
Good	1.35 - 1.25	1.25 - 1.15	1.15 - 1.00	1.00
Fair	1.25 - 1.15	1.15 - 1.05	1.00 - 0.80	0.80
Poor	1.15 - 1.05	1.05 - 0.80	0.80 - 0.60	0.60
Very Poor	1.05 - 0.95	0.95 - 0.75	0.75 - 0.40	0.40

TABLE 6-2
QUALITY OF PAVEMENT DRAINAGE
(from AASHTO, 1993)

Quality of Drainage	Water Removed Within
Excellent	2 hours
Good	1 day
Fair	1 week
Poor	1 month
Very Poor	(water will not drain)

TABLE 6-3
RECOMMENDED VALUES OF DRAINAGE COEFFICIENT, C_d,
FOR PAVEMENT DESIGN
(from AASHTO, 1993)

Quality of Drainage	Percent of Time Pavement Structure is Exposed to Moisture Levels Approaching Saturation			
	< 1 %	1 to 5 %	5 to 25 %	> 25 %
Excellent	1.25 - 1.20	1.20 - 1.15	1.15 - 1.10	1.10
Good	1.20 - 1.15	1.15 - 1.10	1.10 - 1.00	1.00
Fair	1.15 - 1.10	1.10 - 1.00	1.00 - 0.90	0.90
Poor	1.10 - 1.00	1.00 - 0.90	0.90 - 0.80	0.80
Very Poor	1.00 - 0.90	0.90 - 0.80	0.80 - 0.70	0.70

6.6 COST CONSIDERATIONS

As previously stated, the design thickness of an AC overlay with a geotextile interlayer should be determined as if the geotextile is not present. The economic justification of geotextile use is then derived from:

- an increase in pavement life; a decrease in pavement maintenance costs; and an increase in pavement serviceability due to retardation and possible reduction of reflection cracks;
- an increased structural capacity due to drier base and subgrade materials; or
- a combination of the above items.

The old pavement surface condition and overall installation play a very important role in the performance of the paving geotextile. The deteriorated pavement should be repaired, including filling joints and cracks and replacing sections with potholes and faults in their base or subgrade. Under favorable conditions, reflection cracks can be *retarded* for approximately 2 to 5 years as compared to the overlay without the paving grade geotextile. **The anticipated life improvement, under favorable conditions, is approximately 100 to 200% that of an overlay of the same design thickness without a geotextile.** Favorable conditions for the use of a paving grade geotextile with pavement repaving include:

- the presence of fatigue-related pavement failure, evidenced by *alligator* cracks;
- pavement cracks no wider than 3 mm; and
- the thickness of the new overlay designed to meet the structural requirements of the pavement.

The economic benefit of the geotextile interlayer functioning as a moisture barrier is currently not quantified in cost analyses. The effect of the geotextile on the quality of drainage might be used to objectively estimate an increase in pavement structural capacity. This increased capacity then can be used to estimate increased pavement life or to design a thinner AC overlay.

These potential economic benefits can be combined for a particular project. Other cost benefits, currently not quantified, include potential improvement of aesthetics and improved ride quality.

Alternatively, some engineers may reduce the overlay thickness based upon an equivalent performance thickness to justify economics. Extensive research conducted by Caltrans (Predoehl, 1989), implies that a geotextile interlayer is equivalent to 30 mm of asphalt concrete for relatively thin (*i.e.*, ≤ 120 mm) overlays that are structurally adequate. A useful rule of thumb, based upon typical in-place costs, is that a geotextile interlayer is roughly equivalent to the cost of about 15 mm of asphalt concrete. Cost of installed geotextile generally decreases with increased quantities and an experienced local installer.

Considerable insight into the economics of overlay design with geotextiles can be gained from historic cost and performance data (Barksdale, 1991). This data may be locally, regionally, nationally, or internationally generated.

A final economic analysis issue is the probability of success. Geotextile interlayers, as well as other rehabilitation techniques, are not always effective in improving pavement performance. Therefore, an estimate of the probability of success should be included in all economic analysis (Barksdale, 1991). The probability for success will obviously increase with thoroughness of rehabilitation design, local experience with geotextile interlayers, and thoroughness of construction inspection.

6.7 SPECIFICATIONS

The following example specification is a combination of the AASHTO M288 (1997) geotextile material specification and its accompanying construction/installation guidelines. The specification was based on the combined experience of the Texas and California Departments of Transportation, which have had the greatest success using geotextiles in pavement overlays.

PAVEMENT OVERLAY GEOTEXTILE SPECIFICATION

(after AASHTO M288, 1997)

1. SCOPE

1.1 *Description.* This specification is applicable to the use of a geotextile, saturated with asphalt cement, between pavement layers. The function of the pavement geotextile is to act as a waterproofing and stress relieving membrane within the pavement structure. This specification is not intended to describe geotextile membrane systems specifically designed for pavement joints and localized (spot) repairs.

2. REFERENCED DOCUMENTS

2.1 *AASHTO Standards*

T88	Particle Size Analysis of Soils
T90	Determining the Plastic Limit and Plasticity Index of Soils
T99	The Moisture-Density Relationships of Soils Using a 2.5 kg Rammer and a 305 mm Drop

2.2 *ASTM Standards*

D 123	Standard Terminology Relating to Textiles
D 276	Test Methods for Identification of Fibers in Textiles
D 4354	Practice for Sampling of Geosynthetics for Testing
D 4439	Terminology for Geosynthetics
D 4632	Test Method for Grab Breaking Load and Elongation of Geotextiles
D 4759	Practice for Determining the Specification Conformance of Geosynthetics
D 4873	Guide for Identification, Storage, and Handling of Geotextiles
D 6140	Test Method for Determining Asphalt Retention of Paving Fabrics Used in Asphalt Paving for Full Width Applications

3. PHYSICAL AND CHEMICAL REQUIREMENTS

3.1 Fibers used in the manufacture of geotextiles and the threads used in joining geotextiles by sewing, shall consist of long chain synthetic polymers, composed of at least 95% by weight polyolefins or polyesters. They shall be formed into a stable network such that the filaments or yarns retain their dimensional stability relative to each other, including selvages.

3.2 *Geotextile Requirements.* The paving geotextile shall meet the requirements of following Table. All numeric values in the following table represent minimum average roll values (MARV) in the weaker principal direction (*i.e.*, average test results of any roll in a lot sampled for conformance or quality assurance testing shall meet or exceed the minimum values).

Paving Geotextile Property Requirements

Property	Test Method	Units	Requirements
Grab Strength	ASTM D 4632	N	450
Mass per Unit Area	ASTM D 3776	gm/m^2	140
Ultimate Elongation	ASTM D 4632	%	≥ 50
Asphalt Retention[1]	ASTM D 6140	l/m^2	[1,2]
Melting Point	ASTM D 276	°C	150

NOTES:

(1) Asphalt required to saturate paving geotextile only. Asphalt retention must be provided in manufacturer certification (refer to Section 4). Value does not indicate the asphalt application rate required for construction. Refer to Section 9 for discussion of asphalt application rate.

(2) Product asphalt retention property must meet the MARV provided by the manufacturer's certification (refer to Section 4).

4. CERTIFICATION

4.1 The Contractor shall provide to the Engineer a certificate stating the name of the manufacturer, product name, style number, chemical composition of the filaments or yarns and other pertinent information to fully describe the geotextile.

4.2 The Manufacturer is responsible for establishing and maintaining a quality control program to assure compliance with the requirements of the specification. Documentation describing the quality control program shall be made available upon request.

4.3 The Manufacturer's certificate shall state that the furnished geotextile meets MARV requirements of the specification as evaluated under the Manufacturer's quality control program. The certificate shall be attested to be a person having legal authority to bind the Manufacturer.

4.4 Either mislabeling or misrepresentation of materials shall be reason to reject those geotextile products.

5. SAMPLING, TESTING, AND ACCEPTANCE

5.1 Geotextiles shall be subject to sampling and testing to verify conformance with this specification. Sampling for testing shall be in accordance with ASTM D 4354. Acceptance shall be based on testing of either conformance samples obtained using Procedure A of ASTM D 4354, or based on manufacturer's certifications and testing of quality assurance samples obtained using Procedure B of ASTM D 4354. A lot size for conformance or quality assurance sampling shall be considered to be the shipment quantity of the given product or a truckload of the given product, whichever is smaller.

5.2 Testing shall be performed in accordance with the methods referenced in this specification for the indicated application. The number of specimens to test per sample is specified by each test method. Geotextile product acceptance shall be based on ASTM D 4759. Product acceptance is determined by comparing the average test results of all specimens within a given sample to the specification MARV. Refer to ASTM D 4759 for more details regarding geotextile acceptance procedures.

6. SHIPMENT AND STORAGE

6.1 Geotextile labeling, shipment, and storage shall follow ASTM D 4873. Product labels shall clearly show the manufacturer or supplier name, style number, and roll number. Each shipping document shall include a notation certifying that the material is in accordance with the manufacturer's certificate.

6.2 Each geotextile roll shall be wrapped with a material that will protect the geotextile from damage due to shipment, water, sunlight, and contaminants. The protective wrapping shall be maintained during periods of shipment and storage.

6.3 During storage, geotextile rolls shall be elevated off the ground and adequately covered to protect them from the following: site construction damage, precipitation, extended ultraviolet radiation including sunlight, chemicals that are strong acids or strong bases, flames including welding sparks, temperatures in excess of 71°C (160°F), and any other environmental condition that may damage the physical property values of the geotextile.

7. MATERIALS

7.1 *Sealant.* The sealant material used to impregnate and seal the geotextile, as well as bond it to both the base pavement and overlay, shall be a paving grade asphalt recommended by the geotextile manufacturer, and approved by the Engineer.

a. Uncut asphalt cements are the preferred sealants; however, cationic and anionic emulsions may be used provided the precautions outlined in Section 9.3 are followed. Cutbacks and emulsions which contain solvents shall not be used.

b. The grade of asphalt cement specified for hot-mix design in each geographic location is generally the most acceptable material.

7.2 *Sand.* Washed concrete sand may be spread over an asphalt saturated geotextile to facilitate movement of equipment during construction or to prevent tearing or delamination of the geotextile. Hot-mix broadcast in front of construction vehicle tire may also be used to serve this purpose. If sand is applied, excess quantities shall be removed from the geotextile prior to placing the surface course.

a. Sand is not usually required. However, ambient temperatures are occasionally sufficiently high to cause bleed-through of the asphalt sealant resulting in undesirable geotextile adhesion to construction vehicle tires.

8. EQUIPMENT

8.1 The asphalt distributor shall be capable of spraying the asphalt sealant at the prescribed uniform application rate. Not streaking, skipping, or dripping will be permitted. The distributor shall also be equipped with a hand spray having a single nozzle and positive shut-off valve.

8.2 Mechanical or manual lay down equipment shall be capable of laying the geotextile smoothly.

8.3 The following miscellaneous equipment shall be provided: stiff bristle brooms or squeegees to smooth the geotextile; scissors or blades to cut the geotextile; brushes for applying asphalt sealant to geotextile overlaps.

8.4 Pneumatic rolling equipment to smooth the geotextile into the sealant, and sanding equipment may be required for certain jobs. Rolling is especially required on jobs where thin lifts or chip seals are being placed. Rolling helps ensure geotextile bond to the adjoining pavement layers in the absence of heat and weight associated with thick lifts of asphaltic pavement.

9. CONSTRUCTION

9.1 Neither the asphalt sealant nor the geotextile shall be placed when weather conditions, in the opinion of the Engineer, are not suitable. Air and pavement temperatures shall be sufficient to allow the asphalt sealant to hold the geotextile in place. For asphalt cements, air temperature shall be 10°C and rising. For asphalt emulsions, air temperature shall be 15°C and rising.

9.2 The surface on which the geotextile is to be placed shall be reasonably free of dirt, water, vegetation, or other debris. Cracks exceeding 3 mm in width shall be filled with a suitable crack filler. Potholes shall be properly repaired as directed by the Engineer. Fillers shall be allowed to cure prior to geotextile placement.

9.3 The specified rate of asphalt sealant application must be sufficient to satisfy the asphalt retention properties of the geotextile, and bond the geotextile and overlay to the old pavement.

NOTE: When emulsions are used, the application rate must be increased to offset water content of the emulsion.

a. Application of the sealant shall be by distributor spray bar, with hand spraying kept to a minimum. Temperature of the asphalt sealant shall be sufficiently high to permit uniform spray pattern. For asphalt cements the minimum temperature shall be 145°C. To avoid damage to the geotextile, however, the distributor tank temperature shall not exceed 160°C.

b. Spray patterns for asphalt emulsion are improved by heating. Temperatures in the 55°C to 70°C range are desirable. A temperature of 70°C shall not be exceeded since higher temperatures may break the emulsion.

c. The target width of asphalt sealant application shall be the geotextile width plus 150 mm. The asphalt sealant shall not be applied any farther in advance of geotextile placement than the distance the contractor can maintain free of traffic.

d. Asphalt spills shall be cleaned from the road surface to avoid flushing and geotextile movement.

e. When asphalt emulsions are used, the emulsion shall be cured prior to placing the geotextile and final wearing surface. This means essentially no moisture remaining.

9.4 The geotextile shall be placed onto the asphalt sealant with minimum wrinkling prior to the time the asphalt has cooled and lost tackiness. As directed by the Engineer, wrinkles or folds in excess of 25 mm shall be slit and laid flat.

a. Brooming and/or pneumatic rolling will be required to maximize geotextile contact with the pavement surface.

b. Overlap of geotextile joints shall be sufficient to ensure full closure of the joint, but should not exceed 150 mm. Transverse joints shall be lapped in the direction of paving to prevent edge pickup by the paver. A second application of asphalt sealant to the geotextile overlaps will be required if in the judgement of the Engineer additional asphalt sealant is needed to ensure proper bonding of the double geotextile layer.

c. Removal and replacement of geotextile that is damaged will be the responsibility of the contractor.

> NOTE: The problems associated with wrinkles are related to the thickness of the asphalt lift being placed over the geotextile. When wrinkles are large enough to be folded over, there usually is not enough asphalt available from the tack coat to satisfy the requirement of multiple layers of geotextile. Therefore, wrinkles should be slit and laid flat. Sufficient asphalt sealant should be sprayed on the top of the geotextile to satisfy the requirement of the lapped geotextile.

> NOTE: In overlapping adjacent rolls of geotextile it is desirable to keep the lapped dimension as small as possible and still provide a positive overlap. If the lapped dimension becomes too large, the problem of inadequate tack to satisfy the two lifts of geotextile and the old pavement may occur. If this problem does occur then additional asphaltic sealant should be added to the lapped areas. In the application of the additional sealant, care should be taken not to apply too much since an excess will cause flushing.

d. Trafficking the geotextile will be permitted for emergency and construction vehicles only.

9.5 Placement of the hot-mix overlay should closely follow geotextile laydown. The temperature of the mix shall not exceed 160°C. In the event asphalt bleeds through the geotextile causing construction problems before the overlay is placed, the affected areas shall be blotted by spreading sand. To avoid movement of, or damage to the seal — coat saturated geotextile, turning of the paver and other vehicles shall be gradual and kept to a minimum.

9.6 Prior to placing a seal coat (or thin overlay such as an open-graded friction course), lightly sand the geotextile at a spread rate of 0.65 to 1 kg per square meter, and pneumatically roll the geotextile tightly into the sealant.

> **ADVISORY — It is recommended that for safety considerations, trafficking of the geotextile should not be allowed. However, if the contracting agency elects to allow trafficking, the following verbiage is recommended: "If approved by the Engineer, the seal-coat saturated geotextile may be opened to traffic for 24 to 48 hours prior to installing the surface course. Warning signs shall be placed which advise the motorist that the surface may be slippery when wet. The signs shall also post the appropriate safe speed. Excess sand shall be broomed from the surface prior to placing the overlay. If, in the judgement of the Engineer, the geotextile lacks tackiness following exposure to traffic, a light tack coat shall be applied prior to the overlay."**

10. METHOD OF MEASUREMENT

10.1 The geotextile shall be measured by the number of square meters computed from the payment lines shown on the plans or from payment lines established in writing by the Engineer. This excludes seam overlaps.

10.2 Asphalt sealant for the paving fabric will be measured by the liter

11. BASIS OF PAYMENT

11.1 The accepted quantities of geotextile shall be paid for per square meter in place.

11.2 The accepted quantities of asphalt sealant for the paving fabric will be paid for at the contract unit price per liter complete in place.

11.3 Payment will be made under:

Pay Item	Pay Unit
Pavement Overlay Geotextile	Square Meter
Asphalt Sealant	liter

6.8 FIELD INSPECTION

Because proper construction procedures are essential for a successful AC overlay project with geotextiles, good field inspection is very important. Prior to construction, the field inspector should review the guidelines in Section 1.7. Most geotextile manufacturers and suppliers will provide technical assistance during the initial stages of a fabric overlay project. This assistance may be particularly beneficial to inexperienced inspectors and contractors.

One construction problem observed in some installations is the placement of insufficient tack coat. Tack coat should be a separate pay item, and field inspection should monitor the quantity of tack coat placed. Monitoring can be done by gauging or by weighing.

6.9 GEOTEXTILE SELECTION

The selected paving grade geotextile must have the ability to absorb and retain the asphalt tack coat to effectively form a waterproofing and stress-relief interlayer. The most common paving grade geotextiles are needlepunched, nonwoven materials, with a mass per unit area of 120 to 200 g/m^2. These types of geotextiles are very porous and have a high asphalt retention property that benefits the waterproofing and stress-reducing properties of the paving geotextile layer. Thinner, heat-bonded geotextiles have also been used. However, a significant variation in constructability and performance has been found between different paving geotextiles.

Although lighter-weight (*i.e.*, 120 to 135 g/m^2) geotextiles have been previously used, both theory and a limited amount of field evidence indicates that geotextile with a greater mass per unit area (and a greater retention of asphalt), perform better than lighter-weight geotextiles by further reducing stress at the tip of the underlying pavement crack (Graf and Werner, 1993; Grzybowska et al., 1993; Walsh, 1993). As a result, the current AASHTO specification requires a minimum of 140 g/m^2. Numerical analysis indicates that at some level of mass per unit area (e.g., > 200 g/m^2), the bonding of the overlay would be reduced due to shear on the geotextile (Grzybowska et al., 1993; Walsh, 1993).

For overlay design, the appropriate paving grade geotextile should be selected with consideration given to pavement conditions, pavement deflection measurements, and the overlay design traffic (EAL), as presented in Table 6-4.

TABLE 6-4
PAVING GRADE GEOTEXTILE SELECTION

Paving Grade Geotextile	Pavement Conditions Rating[1]	Deflections (mm)	Design Traffic (EAL)
Lighter grade[2] (~140 g/m^2)	65 - 80	< 2.5	≤ 50,000
Medium grade (~170 g/m^2)	40 - 50	> 1.5	≤ 2,000,000
Class 3 (heaviest grade) (~200 g/m^2)	20 - 30	> 1.5	> 2,000,000

NOTES:
1. From The Asphalt Institute (1983), and adopted from TRB Record No. 700 -
   - 65 - 80: Fairly good, slight longitudinal and alligator cracking. Few slightly rough and uneven.
   - 40 - 50: Poor to fair, moderate longitudinal and alligator crackings. Surface is slightly rough and uneven.
   - 20 - 30: Poor conditions with extensive alligator and longitudinal crackings. Surface is very rough and uneven.

6.10 RECYCLING

AC overlays used with a geotextile can be recycled. The most common practice is to mill down to just above the geotextile interlayer. This process maintains the benefits of the interlayer when the recycled overlay is replaced (Marienfeld and Smiley, 1994). Alternatively, milling may include the geotextile interlayer. A detailed study of recycling a nonwoven polypropylene geotextile (Christman, 1981) concluded that overlay with geotextile interlayers does not pose any problem to the milling operation. Additionally, no apparent differences were noted in the properties of dryer-drum recycled mixtures (50-50 blend) containing or lacking geotextiles.

6.11 OTHER GEOSYNTHETIC MATERIALS

6.11-1 Membrane Strips

A variety of commercially available, heavy-duty membrane strips are used over cracks and joints of PCC pavements that are overlaid with AC. Typically, these materials are composites of woven or nonwoven geotextiles and modified asphalt membranes. Materials of single-layer geotextiles with rubber-asphalt membranes are typically used for strip waterproofing. Materials of double-layer geotextiles that sandwich a modified asphalt membrane are typically used to reduce and retard reflective cracking. Crack reduction interlayers are typically 3.5 mm thick and are capable of maintaining 95% of their thickness during installation and in-service use. Interlayer strips are typically 0.3 to 1 m wide, and usually weigh 1600 to 3300 g/m^2 -- which is heavier than the typical geotextile interlayer that weighs about 1300 g/m^2 with asphalt impregnation (Barksdale, 1991).

The installation of heavy-duty membranes is relatively easy. Usually the manufacturer's installation recommendations are followed because of the wide variation of products and installation requirements. Single-, two-, or three-step installation processes are required for the various products.

Advantages of strips include:

- limited area of installation, and, therefore, less potential installation problems;
- factory-applied asphalt, and, therefore, less field variances; and
- heavier weight, and possible function as a stress-relieving membrane interlayer of the material.

The moderate amount of documented field performance data developed to date has been summarized by Barksdale (1991).

6.11-2 Geogrids

High-strength and high-stiffness polymer and fiberglass geogrids have been used on a relatively limited basis for full-width and strip overlay applications. These grids primarily function as a reinforcing element, provided they are sufficiently stiff. The limited use of geogrids in this application shows promise. However, additional well-planned, monitored installations are needed.

6.11-3 Geocomposites

Geogrid-geotextile composites are available; however, there is limited experience with these products to date. The intent of such a composite is to have a material that installs similarly to geotextile overlays. A properly installed composite should function as stress-relieving interlayer, moisture barrier, and reinforcement (provided the geogrid has a high modulus).

6.12 REFERENCES

References quoted within this section are listed below. *Key* references are noted in bold type.

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7.0 REINFORCED EMBANKMENTS ON SOFT FOUNDATIONS

7.1 BACKGROUND

Embankments constructed on soft foundation soils have a tendency to spread laterally because of horizontal earth pressures acting within the embankment. These earth pressures cause horizontal shear stresses at the base of the embankment which must be resisted by the foundation soil. If the foundation soil does not have adequate shear resistance, failure can result. Properly designed horizontal layers of high-strength geotextiles or geogrids can provide reinforcement which increase stability and prevent such failures. Both materials can be used equally well, provided they have the requisite design properties. There are some differences in how they are installed, especially with respect to seaming and field workability. Also, at some very soft sites, especially where there is no root mat or vegetative layer, geogrids may require a lightweight geotextile separator to provide filtration and prevent contamination of the first lift if it is an open-graded or similar type soil. A geotextile is not required beneath the first lift if it is sand, which meets soil filtration criteria.

The reinforcement may also reduce horizontal and vertical displacements of the underlying soil and thus reduce differential settlement. *It should be noted that the reinforcement will not reduce the magnitude of long-term consolidation or secondary settlement of the embankment.*

The use of reinforcement in embankment construction may allow for:

- an increase in the design factor of safety;
- an increase in the height of the embankment;
- a reduction in embankment displacements during construction, thus reducing fill requirements; and
- an improvement in embankment performance due to increased uniformity of post-construction settlement.

This chapter assumes that all the common foundation treatment alternatives for the stabilization of embankments on soft or problem foundation soils have been carefully considered during the preliminary design phase. Holtz (1989) discusses these treatment alternatives and provides guidance about when embankment reinforcement is feasible. In some situations, the most economical final design may be some combination of a conventional foundation treatment alternative together with geosynthetic reinforcement. Examples include preloading and stage construction with prefabricated (*wick*) vertical drains, the use of stabilizing berms, or pile-supported bridge approach embankments - each used with geosynthetic reinforcement at the base of the embankment.

7.2 APPLICATIONS

R inforced embankments over weak foundations typically fall into one of two situations - construction over uniform deposits, and construction over local anomalies (Bonaparte, Holtz, and Giroud, 1985). The more common is embankments, dikes, or levees constructed over very soft, saturated silt, clay, or peat layers (Figure. 7-1a). In this situation, the reinforcement is usually placed with its strong direction perpendicular to the centerline of the embankment, and plane strain conditions are assumed to prevail. Additional reinforcement with its strong direction oriented parallel to the centerline may also be required at the ends of the embankment.

The second reinforced embankment situation includes foundations below the embankment that are locally weak or contain voids. These zones or voids may be caused by sink holes, thawing ice (thermokarsts), old stream beds, or pockets of silt, clay, or peat (Figure 7-1b). In this application, the role of the reinforcement is to bridge over the weak zones or voids, and tensile reinforcement may be required in more than one direction. Thus, the strong direction of the reinforcing must be placed in proper orientation with respect to the embankment centerline (Bonaparte and Christopher, 1987).

Geotextiles may also be used as separators for displacement-type embankment construction (Holtz, 1989). In this application, the geotextile does not provide any reinforcement but only acts as a separator to maintain the integrity of the embankment as it displaces the subgrade soils. In this case, geotextile design is based upon constructability and survivability, and a high elongation material may be selected.

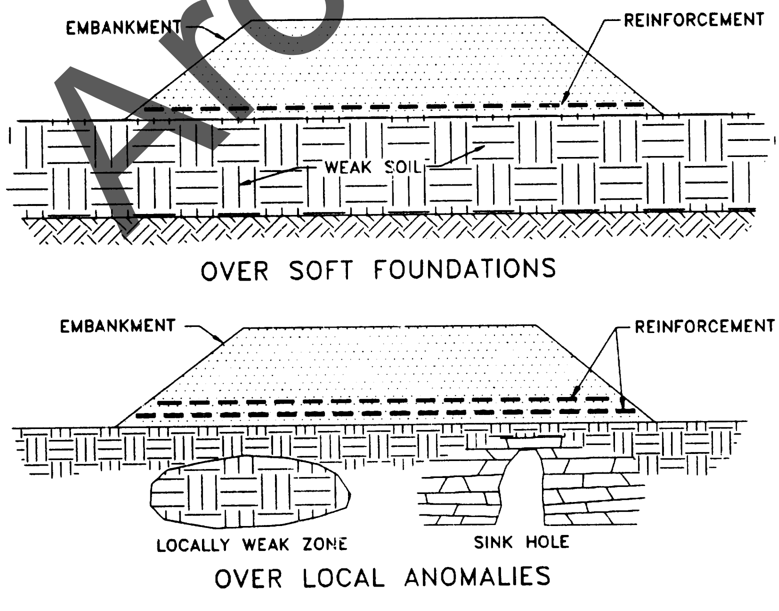


Figure 7-1 Reinforced embankment applications (after Bonaparte and Christopher, 1987).

7.3 DESIGN GUIDELINES FOR REINFORCED EMBANKMENTS ON SOFT SOILS

7.3-1 Design Considerations

As with ordinary embankments on soft soils, the basic design approach for reinforced embankments is to design against failure. The ways in which embankments constructed on soft foundations can fail have been described by Terzaghi and Peck (1967); Haliburton, Anglin and Lawmaster (1978 a and b); Fowler (1981); Christopher and Holtz (1985); and Koerner (1990), among others. Figure 7-2 shows unsatisfactory behavior that can occur in reinforced embankments. The three possible modes of failure indicate the types of stability analyses that are required. In addition, settlement of the embankment and potential creep of the reinforcement must be considered, although creep is only a factor if the creep rate in the reinforcement is greater than the strength gain occurring in the foundation due to consolidation. Because the most critical condition for embankment stability is at the end of construction, the reinforcement only has to function until the foundation soils gain sufficient strength to support the embankment.

The calculations required for stability and settlement utilize conventional geotechnical design procedures modified only for the presence of the reinforcement.

The stability of an embankment over soft soil is usually determined by the *total stress* method of analysis, which is conservative since the analysis generally assumes that no strength gain occurs in the compressible soil. The stability analyses presented in this text uses the *total stress* approach, because it is simple and appropriate for reinforcement design (Holtz, 1989).

It is always possible to calculate stability in terms of the effective stresses using the *effective stress* shear strength parameters. However, this calculation requires an accurate estimate of the field pore pressures to be made during the project design phase. Additionally, high-quality, undisturbed samples of the foundation soils must be obtained and K_o consolidated-undrained triaxial tests conducted in order to obtain the required design soil parameters. Because the prediction of in situ pore pressures in advance of construction is not easy, it is essential that field pore pressure measurements using high quality piezometers be made during construction to control the rate of embankment filling. Preloading and staged embankment construction are discussed in detail by Ladd (1991). Note that by taking into account the strength gain that occurs with staged embankment construction, lower strength and therefore lower cost reinforcement can be utilized. However; the time required for construction may be significantly increased and the costs of the site investigation, laboratory testing, design analyses, field instrumentation, and inspection are also greater.

The *total stress* design steps and methodology are detailed in the following section, because they are more appropriate for reinforcement design (Holtz, 1989).

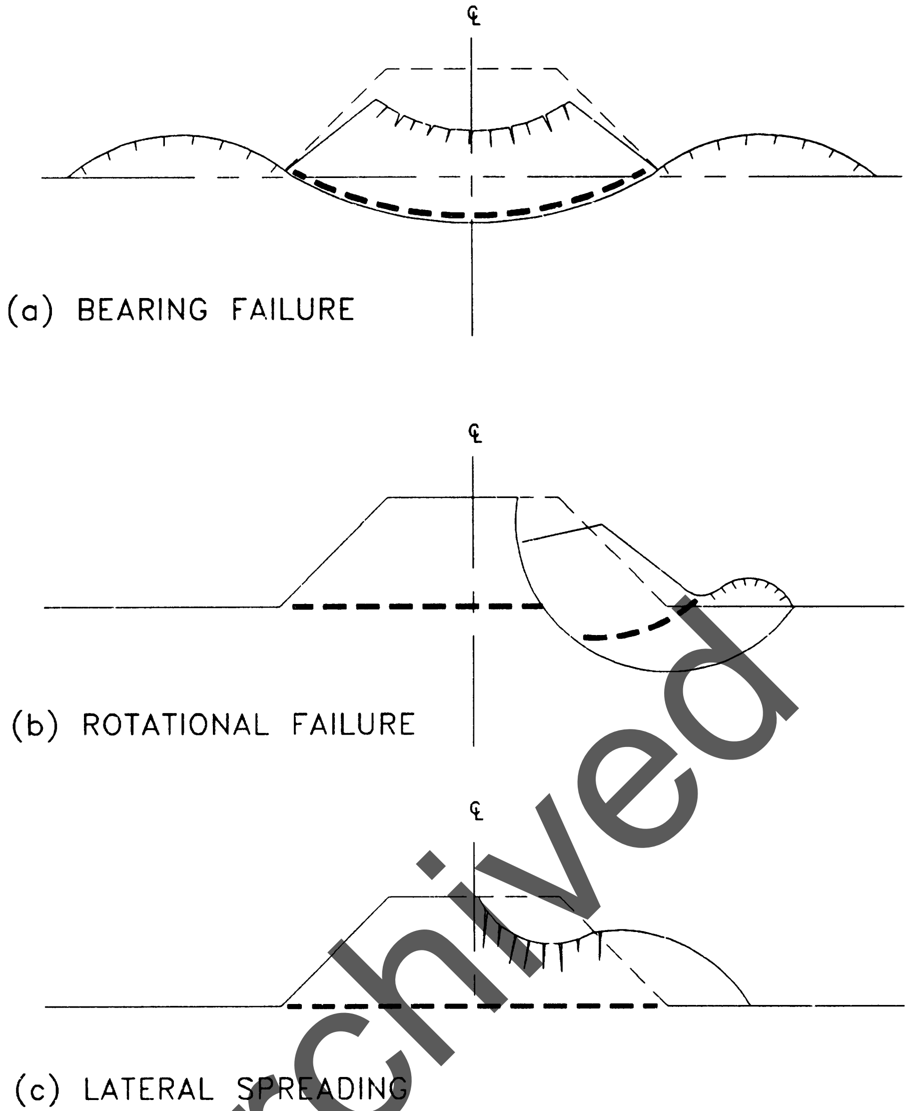


Figure 7-2 Reinforced embankments failure modes (after Haliburton et al., 1978).

7.3-2 Design Steps

The following is a step-by-step procedure for design of reinforced embankments. Additional comments on each step can be found in Section 7.3-3.

STEP 1. Define embankment dimensions and loading conditions.

A. Embankment height, H

B. Embankment length

C. Width of crest

D. Side slopes, b/H

E. External loads
   1. surcharges
   2. temporary (traffic) loads
   3. dynamic loads

F. Environmental considerations
   1. frost action
   2. shrinkage and swelling
   3. drainage, erosion, and scour

G. Embankment construction rate
   1. project constraints
   2. anticipated or planned rate of construction

STEP 2. Establish the soil profile and determine the engineering properties of the foundation soil.

A. From a subsurface soils investigation, determine
   1. subsurface stratigraphy and soil profile
   2. groundwater table (location, fluctuation)

B. Engineering properties of the subsoils
   1. Undrained shear strength, c_u, for end of construction
   2. Drained shear strength parameters, c' and ϕ', for long-term conditions
   3. Consolidation parameters (C_c, C_r, c_v, σ_p')
   4. Chemical and biological factors that may be detrimental to the reinforcement

C. Variation of properties with depth and areal extent

STEP 3. Obtain engineering properties of embankment fill materials.

A. Classification properties

B. Moisture-density relationships

C. Shear strength properties

D. Chemical and biological factors that may be detrimental to the reinforcement

STEP 4. Establish minimum appropriate factors of safety and operational settlement criteria for the embankment. Suggested minimum factors of safety are as follows.

A. Overall bearing capacity: 1.5 to 2

B. Global (rotational) shear stability at the end of construction: 1.3

C. Internal shear stability, long-term: 1.5

D. Lateral spreading (sliding): 1.5

E. Dynamic loading: 1.1

F. Settlement criteria: dependent upon project requirements

STEP 5. Check bearing capacity.

A. When the thickness of the soft soil is much greater than the width of the embankment, use classical bearing capacity theory:

$$q_{ult} = \gamma_{fill} H = c_u N_c \qquad [7\text{-}1]$$

where N_c, the bearing capacity factor, is usually taken as 5.14 -- the value for a strip footing on a cohesive soil of constant undrained shear strength, c_u, with depth. This approach underestimates the bearing capacity of reinforced embankments, as discussed in Section 7.3-3.

B. When the soft soil is of limited depth, perform a *lateral squeeze* analysis (Section 7.3-3).

STEP 6. Check rotational shear stability.

Perform a rotational slip surface analysis on the unreinforced embankment and foundation to determine the critical failure surface and the factor of safety against local shear instability.

A. If the calculated factor of safety is greater than the minimum required, then reinforcement is not needed. Check lateral embankment spreading (Step 7).

B. If the factor of safety is less than the required minimum, then calculate the required reinforcement strength, T_g, to provide an adequate factor of safety using Figure 7-3 or alternative solutions (Section 7.3-3), where:

$$T_g = \frac{FS\ (M_D\ ) - M_R}{R\ \cos(\theta - \beta)}$$

STEP 7. Check lateral spreading (sliding) stability.

Perform a lateral spreading or sliding wedge stability analysis (Figure 7-4).

A. If the calculated factor of safety is greater than the minimum required, then reinforcement is not needed for this failure possibility.

B. If the factor of safety is inadequate, then determine the lateral spreading strength of reinforcement, T_{ls}, required -- see Figure 7-4b. Soil/geosynthetic cohesion, C_a, should be assumed equal to 0 for extremely soft soils and low embankments. A cohesion value should be included with placement of the second and subsequent fills in staged embankment construction.

C. Check sliding above the reinforcement. See Figure 7-4a.

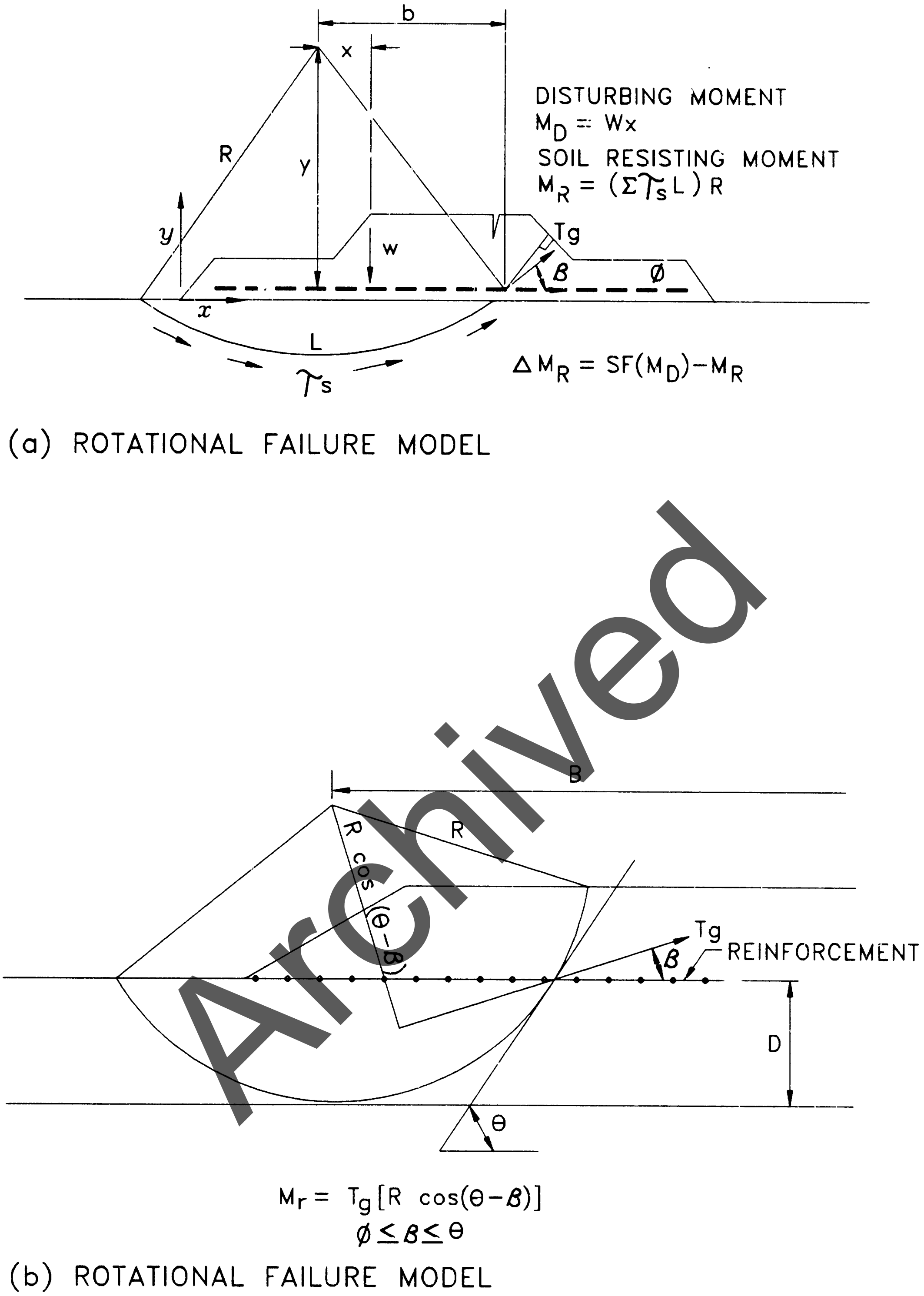


Figure 7-3 Reinforcement required to provide rotational stability: (a) Christopher and Holtz (1985) after Wager (1981); (b) Bonaparte and Christopher (1987) for the case in which the reinforcement does not increase soil strength.

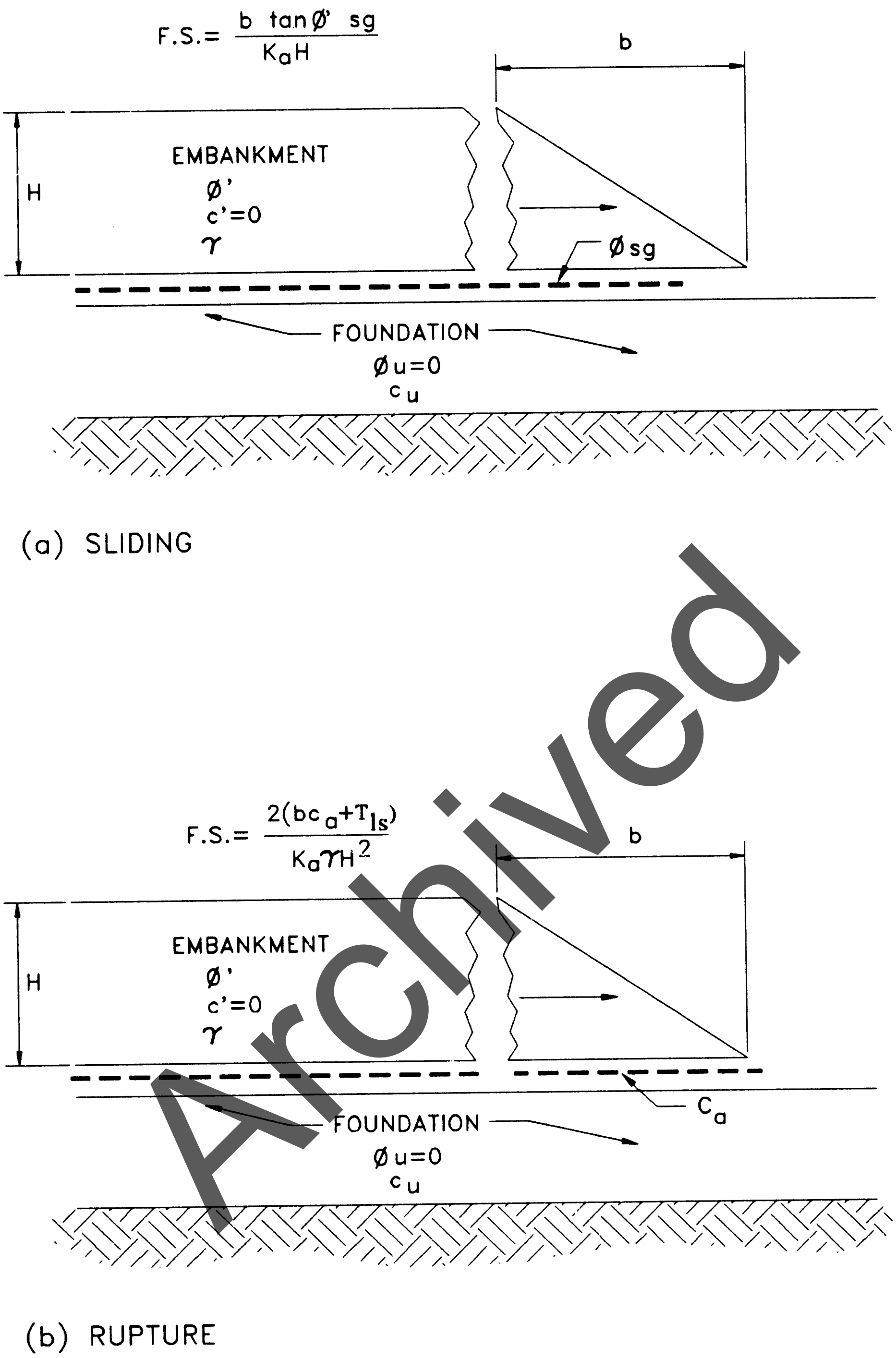


Figure 7-4 Reinforcement required to limit lateral embankment spreading: (a) embankment sliding on reinforcement; (b) rupture of reinforcement and embankment sliding on foundation soil (from Bonaparte and Christopher, 1987).

STEP 8. Establish tolerable geosynthetic deformation requirements and calculate the required reinforcement modulus, J, based on wide width (ASTM D 4595) tensile testing.

Reinforcement Modulus: $J = T_{ls} / \epsilon_{geosynthetic}$ [7-2]

Recommendations for strain limits, based on type of fill soil materials and for construction over peats, are:

Cohesionless soils:	$\epsilon_{geosynthetic} = 5 \text{ to } 10\%$	[7-3]
Cohesive soils:	$\epsilon_{geosynthetic} = 2\%$	[7-4]
Peats:	$\epsilon_{geosynthetic} = 2 \text{ to } 10\%$	[7-5]

STEP 9. Establish geosynthetic strength requirements in the embankment's longitudinal direction (i.e., direction of the embankment alignment).

A. Check bearing capacity and rotational slope stability at the ends of the embankment (Steps 5 and 6).

B. Use strength and elongation determined from Steps 7 and 8 to control embankment spreading during construction and to control bending following construction.

C. As the strength of the seams transverse to the embankment alignment control strength requirements, seam strength requirements are the higher of the strengths determined from Steps 9.A or 9.B.

STEP 10. Establish geosynthetic properties (Section 7.4).

A. Design strengths and modulus are based on the ASTM D 4595 wide width tensile test. This test standard permits definition of tensile modulus in terms of: (i) initial tensile modulus; (ii) offset tensile modulus; or (iii) secant tensile modulus. Furthermore, the secant modulus may be defined between any two strain points. Geosynthetic modulus for design of embankments should be determined using a secant modulus, defined with the zero strain point and design strain limit (i.e., 2 to 10%) point.

B. Seam strength is quantified with ASTM D 4884 test method, and is equal to the strength required in the embankment's longitudinal direction.

C. Soil-geosynthetic friction, ϕ_{sg}, based on ASTM D 5321 with on-site soils. For preliminary estimates, assume $\phi_{sg} = 2/3\phi$; for final design, testing is recommended.

D. Geotextile stiffness based on site conditions and experience. See Section 7.4-5.

E. Select survivability and constructability requirements for the geosynthetic based on site conditions, backfill materials, and equipment, using Tables 7-1, 7-2, and 7-3.

STEP 11. Estimate magnitude and rate of embankment settlement.

Use conventional geotechnical procedures and practices for this step.

STEP 12. Establish construction sequence and procedures.

See Section 7.8.

STEP 13. Establish construction observation requirements.

See Sections 7.8 and 7.9.

STEP 14. Hold preconstruction meetings.

Consider a *partnering* type contract with a disputes resolution board.

STEP 15. Observe construction and build with confidence (if the procedures outlined in these guidelines are followed!)

7.3-3 Comments on the Design Procedure

STEPS 1 and 2 need no further elaboration.

STEP 3. Obtain embankment fill properties.

Follow traditional geotechnical practice, except that the first few lifts of fill material just above the geosynthetic should be free-draining granular materials. This requirement provides the best frictional interaction between the geosynthetic and fill, as well as providing a drainage layer for excess pore water to dissipate from the underlying soils. Other fill materials may be used above this layer as long as the strain compatibility of the geosynthetic is evaluated with respect to the backfill materials (Step 8).

STEP 4. Establish design factors of safety.

The minimum factors of safety previously stated are recommended for projects with modern state-of-the-practice geotechnical site investigations and laboratory testing. Those factors may be adjusted depending on the method of analysis, type and use of facility being designed, the known conditions of the subsurface, the quality of the samples and soils testing, the cost of failure, the probability of unusual events occurring, and the engineer's previous experience on similar projects and sites. In short, all of the uncertainties in loads, analyses, and soil properties influence the choice of appropriate factors of safety. Typical factors of safety for unreinforced embankments also seem to be appropriate for reinforced embankments.

When the calculated factor of safety is greater than 1 but less than the minimum allowable factor of safety for design, say 1.3 or 1.5, then the geosynthetic provides an additional factor of safety or a *second line of defense* against failure. On the other hand, when the calculated factor of safety for the unreinforced embankment is significantly less than 1, the geosynthetic reinforcement is the difference between success and failure. In this latter case, **construction considerations (Section 7.8) become crucial to the project success**.

Maximum tolerable post-construction settlement and embankment deformations, which depend on project requirements, must also be established.

STEP 5. Check overall bearing capacity.

Reinforcement does not increase the overall bearing capacity of the foundation soil. If the foundation soil cannot support the weight of the embankment, then the embankment cannot be built. Thus, the overall bearing capacity of the entire embankment must be satisfactory before considering any possible reinforcement. As such, the vertical stress due to the embankment can be treated as an average stress over the entire width of the embankment, similar to a semi-rigid mat foundation.

The bearing capacity can be calculated using classical soil mechanics methods (Terzaghi and Peck, 1967; Vesic, 1975; Perloff and Baron, 1976; and U.S. Navy, 1982) which use limiting equilibrium-type analyses for strip footings, assuming logarithmic spiral failure surfaces on an infinitely deep foundation. These analyses are not appropriate if the thickness of the underlying soft deposit is small compared to the width of the embankment. In this case, high lateral stresses in the confined soft stratum beneath the embankment could lead to a *lateral squeeze*-type failure. The shear forces developed under the embankment should be compared to the corresponding shear strength of the soil. Approaches discussed by Jürgenson (1934), Silvestri (1983), and Bonaparte, Holtz and Giroud (1985), Rowe and Soderman (1987a), Hird and Jewell (1990), and Humphrey and Rowe (1991) are appropriate. The designer should be aware that the analysis for lateral squeeze is only approximate, and no single method is completely accepted by geotechnical engineers at present.

In a review of 40 reinforced embankment case histories, Humphrey and Holtz (1986) and Humphrey (1987) found that in many cases, the failure height predicted by classical bearing capacity theory was significantly less than the actual constructed height, especially if high strength geotextiles and geogrids were used as the reinforcement. Figure 7-5 shows the embankment height versus average undrained shear strength of the foundation. Significantly, four embankments failed at heights 2 m greater than predicted by Equation 7-1 (line B in Figure 7-5). The two reinforced embankments that failed below line B were either on peat or under reinforced (Humphrey, 1987). It appears that in many cases, the reinforcement enhances the beneficial effect the following factors have on stability:

- limited thickness or increasing strength with depth of the soft foundation soils (Rowe and Soderman, 1987 a and b; Jewell, 1988);
- the dry crust (Humphrey and Holtz, 1989);
- flat embankment side slopes (*e.g.*, Humphrey and Holtz, 1987); or
- dissipation of excess pore pressures during construction.

If the factor of safety for bearing capacity is sufficient, then continue with the next step. If not, consider increasing the embankment's width, flattening the slopes, adding toe berms, or improving the foundation soils by using stage construction and drainage enhancement or other alternatives, such as relocating the alignment or placing the roadway on an elevated structure.

STEP 6. Check rotational shear stability.

The next step is to calculate the factor of safety against a circular failure through the embankment and foundation using classical limiting equilibrium-type stability analyses. If the factor of safety does not meet the minimum design requirements (Step 4), then the reinforcing tensile force required to increase the factor of safety to an acceptable level must be estimated.

This is done by assuming that the reinforcement acts as a stabilizing tensile force at its intersection with the slip surface being considered. The reinforcement thus provides the additional resisting moment required to obtain the minimum required factor of safety. The analysis is shown in Figure 7-3.

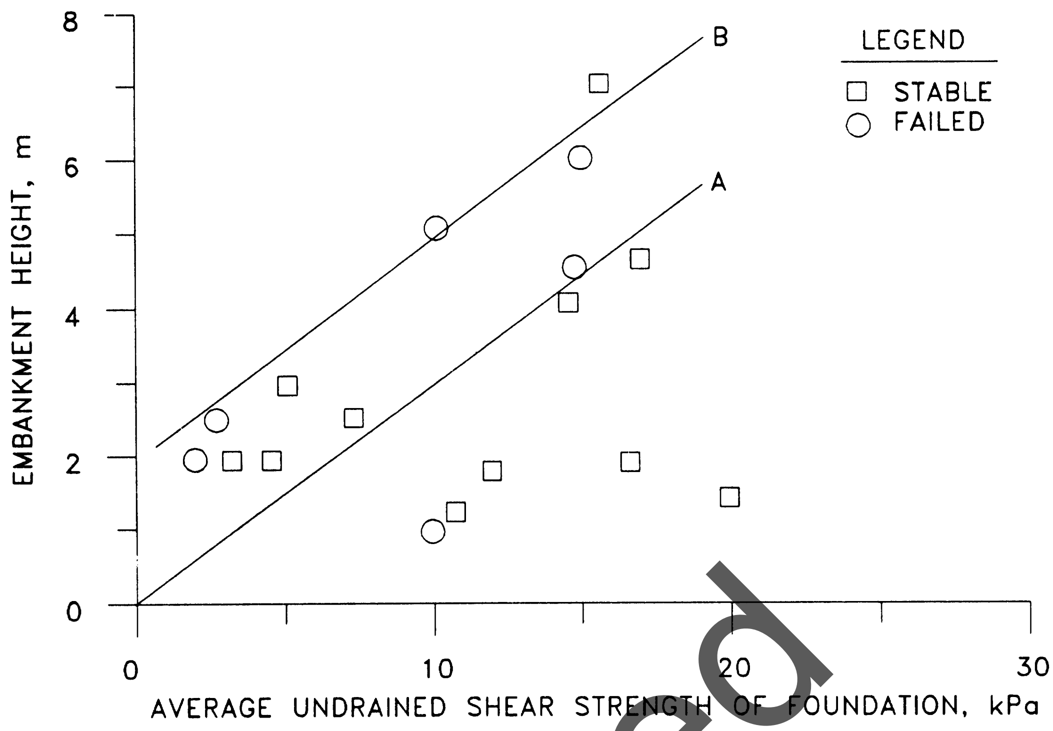


Figure 7-5 Embankment height versus undrained shear strength of foundation; line A: classical bearing capacity theory (Eq. 7-1); line B: line A + 2 m (after Humphrey, 1987).

The analysis consists of determining the most critical failure surface(s) using conventional limiting equilibrium analysis methods. For each critical sliding surface, the driving moment (M_D) and soil resisting moment (M_R) are determined as shown in Figure 7-3a. The additional resisting moment ΔM_R to provide the required factor of safety is calculated as shown in Figure 7-3b. Then one or more layers of geotextiles or geogrids with sufficient tensile strength at tolerable strains (Step 7) are added at the base of the embankment to provide the required additional resisting moment. If multiple layers are used, they must be separated by a granular layer and they must have compatible stress-strain properties (*e.g.*, the same type of reinforcement must be used for each layer).

A number of procedures have been proposed for determining the required additional reinforcement, and these are summarized by Christopher and Holtz (1985), Bonaparte and Christopher (1987), Holtz (1990), and Humphrey and Rowe (1991). The basic difference in the approaches is in the assumption of the reinforcement force orientation at the location of the critical slip surface (the angle ß in Figures 7-3a and 7-3b). It is conservative to assume that the reinforcing force acts horizontally at the location of the reinforcement ($\beta = 0$). In this case, the additional reinforcing moment is equal to the required geosynthetic strength, T_g, times the vertical distance, y, from the plane of the reinforcement to the center of rotation, or:

$$\Delta M_R = T_g\, y \qquad [7\text{-}6a]$$

as determined for the most critical failure surface, shown in Figure 7-3a. This approach is conservative because it neglects any possible reinforcement reorientation along the alignment of the failure surface, as well as any confining effect of the reinforcement.

A less-conservative approach assumes that the reinforcement bends due to local displacements of the foundation soils at the onset of failure, with the maximum possible reorientation located tangent to the slip surface ($\beta = \theta$ in Figure 7-3b). In this case,

$$\Delta M_R = T_g\,[R \cos(\theta - \beta)] \qquad [7\text{-}6b]$$

where,

θ = angle from horizontal to tangent line as shown in Figure 7-3.

Limited field evidence indicates that it is actually somewhere in between the horizontal and tangential (Bonaparte and Christopher, 1987) depending on the foundation soils, the depth of soft soil from the original ground line in relation to the width of the embankment (D/B ratio), and the stiffness of the reinforcement. Based on the minimal information available, the following suggestions are provided for selecting the orientation:

- $\beta = 0$ for brittle, strain-sensitive foundations soils (*e.g.*, leached marine clays) or where a crust layer is considered in the analysis for increased support;
- $\beta = \theta/2$ for D/B < 0.4 and moderate to highly compressible soils (*e.g.*, soft clays, peats);
- $\beta = \theta$ for D/B ≥ 0.4 highly compressible soils (*e.g.*, soft clays, peats); and reinforcement with high elongation potential ($\epsilon_{design} \geq 10\%$), and large tolerable deformations; and
- $\beta = 0$ when in any doubt!

Other approaches, as discussed by Bonaparte and Christopher (1987), require a more rigorous analysis of the foundation soils deformation characteristics and the reinforcement strength compatibility.

In each method, the depth of the critical failure surface must be relatively shallow, *i.e.*, y in Figure 7-3a must be large, otherwise the geosynthetic contribution toward increasing the resisting moment will be small. On the other hand, Jewell (1988) notes that shallow slip surfaces tend to underestimate the driving force in the embankment, and both he and Leshchinsky (1987) have suggested methods to address this problem.

STEP 7. Check lateral spreading (sliding) stability.

A simplified analysis for calculating the reinforcement required to limit lateral embankment spreading is illustrated in Figure 7-4. For unreinforced as well as reinforced embankments, the driving forces result from the lateral earth pressures developed within the embankment and which must, for equilibrium, be transferred to the foundation by shearing stresses (Holtz, 1990). Instability occurs in the embankment when either:

1. the embankment slides on the reinforcement (Figure 7-4a); or
2. the reinforcement fails in tension and the embankment slides on the foundation soil (Figure 7-4b).

In the latter case, the shearing resistance of the foundation soils just below the embankment is insufficient to maintain equilibrium. Thus, in both cases, the reinforcement must have sufficient friction to resist sliding on the reinforcement plane, and the geosynthetic tensile strength must be sufficient to resist rupture as the potential sliding surface passes through the reinforcement.

The forces involved in the analysis of embankment spreading are shown in Figure 7-4 for the two cases above. The lateral earth pressures, usually assumed to be active, are of a maximum at the crest of the embankment. The factor of safety against embankment spreading is found from the ratio of the resisting forces to the actuating (driving) forces. The recommended factor of safety against sliding is 1.5 (Step 4). If the required soil-geosynthetic friction angle is greater than that reasonably achieved with the reinforcement, embankment soils and subgrade, then the embankment slopes must be flattened or berms must be added. Sliding resistance can be increased by the soil improvement techniques mentioned above. Generally, however, there is sufficient frictional resistance between geotextiles and geogrids commonly used for reinforcement and granular fill. If this is the case, then the resultant lateral earth pressures must be resisted by the tension in the reinforcement.

STEP 8. Establish tolerable deformation requirements for the geosynthetic.

Excessive deformation of the embankment and its reinforcement may limit its serviceability and impair its function, even if total collapse does not occur. Thus, an analysis to establish deformation limits of the reinforcement must be performed. The most common way to limit deformations is to limit the allowable strain in the geosynthetic. This is done because the geosynthetic tensile forces required to prevent failure by lateral spreading are not developed without some strain, and some lateral movement must be expected. Thus, geosynthetic modulus is used to control lateral spreading (Step 7). The distribution of strain in the geosynthetic is assumed to vary linearly from zero at the toe to a maximum value beneath the crest of the embankment. This is consistent with the development of lateral earth pressures beneath the slopes of the embankment.

For the assumed linear strain distribution, the maximum strain in the geosynthetic will be equal to twice the average strain in the embankment. Fowler and Haliburton (1980) and Fowler (1981) found that an average lateral spreading of 5% was reasonable, both from a construction and geosynthetic property standpoint. If 5% is the average strain, then the maximum expected strain would be 10%, and the geosynthetic modulus would be determined at 10% strain (Equation 7-3). However, it has been suggested that a modulus at 10% strain would be too large, and that smaller maximum values at, say 2 to 5%, are more appropriate.

If cohesive soils are used in the embankment, then the modulus should be determined at 2% strain to reduce the possibility of embankment cracking (Equation 7-4). Of course, if embankment cracking is not a concern, then these limiting reinforcement strain values could be increased. Keep in mind, however, that if cracking occurs, no resistance to sliding is provided. Further, the cracks could fill with water, which would add to the driving forces.

Additional discussion of geosynthetic deformation is given in Christopher and Holtz (1985 and 1989), Bonaparte, Holtz and Giroud (1985), Rowe and Mylleville (1989 and 1990), and Humphrey and Rowe (1991).

STEP 9. Establish geosynthetic strength requirements in the longitudinal direction.

Most embankments are relatively long but narrow in shape. Thus, during construction, stresses are imposed on the geosynthetic in the longitudinal direction, *i.e.*, in the along direction the centerline. Reinforcement may be also required for loadings that occur at bridge abutments, and due to differential settlements and embankment bending, especially over nonuniform foundation conditions and at the edges of soft soil deposit.

Because both sliding and rotational failures are possible, analyses procedures discussed in Steps 6 and 7 should be applied, but in the direction along the alignment of the embankment. This determines the longitudinal strength requirements of the geosynthetic. Because the usual placement of the geosynthetic is in strips perpendicular to the centerline, the longitudinal stability will be controlled by the strength of the transverse seams.

STEP 10. Establish geosynthetic properties.

See Section 7.4 for a determining the required properties of the geosynthetic.

STEP 11. Estimate magnitude and rate of embankment settlement.

Although not part of the stability analyses, both the magnitude and rate of settlement of the embankment should be considered in any reinforcement design. There is some evidence from finite element studies that differential settlements may be reduced somewhat by the presence of geosynthetic reinforcement. Long-term or consolidation settlements are **not** influenced by the geosynthetic, since compressibility of the foundation soils is not altered by the reinforcement, although the stress distribution may be somewhat different. Present recommendations provide for reinforcement design as outlined in Steps 6 - 10 above. Then use conventional geotechnical methods to estimate immediate, consolidation, and secondary settlements, as if the embankment was unreinforced (Christopher and Holtz, 1985).

Possible creep of reinforced embankments on soft foundations should be considered in terms of the geosynthetic creep rate versus the consolidation rate and strength gain of the foundation. If the foundation soil consolidates and gains strength at a rate faster than (or equal to) the rate the geosynthetic loses strength due to creep, there is no problem. Many soft soils such as peats, silts and clays with sand lenses have high permeability, therefore, they gain strength rapidly, but each case should be analyzed individually.

STEP 12. Establish construction sequence and procedures.

The importance of proper construction procedures for geosynthetic reinforced embankments on very soft foundations cannot be over emphasized. A specific construction sequence is usually required to avoid failures during construction.

See Section 7.8 for details on site preparation, special construction equipment, geosynthetic placement procedures, seaming techniques, and fill placement and compaction procedures.

STEP 13. Establish construction observation requirements

See Sections 7.8 and 7.9.

A. Instrumentation. As a minimum, install piezometers, settlement points, and surface survey monuments. Also consider inclinometers to observe lateral movement with depth.

   Note that the purpose of the instrumentation in soft ground reinforcement projects is not for research but to verify design assumptions and to control and, usually, expedite construction.

B. Geosynthetic inspection. Be sure field personnel understand:
- geosynthetic submittal for acceptance prior to installation;
- testing requirements;
- fill placement procedures; and
- seam integrity verification.

STEP 14. Hold preconstruction meetings

It has been our experience that the more potential contractors know about the overall project, the site conditions, and the assumptions and expectations of the designers, the more realistically they can bid; and, the project is more successful. Prebid and preconstruction information meetings with contractors have been very successful in establishing a good, professional working relationship between owner, design engineer, and contractor. *Partnering* type contracts and a disputes resolution board can also be used to reduce problems, claims, and litigation.

STEP 15. Observe construction

Inspection should be performed by a trained and knowledgeable inspector, and good documentation of construction should be maintained.

7.4 SELECTION OF GEOSYNTHETIC AND FILL PROPERTIES

Once the design strength requirements have been established, the appropriate geosynthetic must be selected. In addition to its tensile and frictional properties, drainage requirements, construction conditions, and environmental factors must also be considered. Geosynthetic properties required for reinforcement applications are given in Table 7-1. The selection of appropriate fill materials is also an important aspect of the design. When possible, granular fill is preferred, especially for the first few lifts above the geosynthetic.

7.4-1 Geotextile and Geogrid Strength Requirements

The most important mechanical properties are the tensile strength and modulus of the reinforcement, seam strength, soil-geosynthetic friction, and system creep resistance.

The tensile strength and modulus values should preferably be determined by an in-soil tensile test. From research by McGown, Andrawes, and Kabir (1982) and others, we know that in-soil properties of many geosynthetics are markedly different than those from tests conducted in air. However, in-soil tests are not yet routine nor standardized, and the test proposed by Christopher,

TABLE 7-1
GEOSYNTHETIC PROPERTIES REQUIRED FOR REINFORCEMENT APPLICATIONS

Criteria and Parameter	Property[1]
Design Requirements:	
a. Mechanical	
Tensile strength	Wide width strength
Tensile modulus	Wide width strength
Seam strength	Wide width strength
Tension creep	Tension creep
Soil-geosynthetic friction	Soil-geosynthetic friction angle
b. Hydraulic	
Piping resistance	Apparent opening size
Permeability	Permeability
Constructability Requirements:	
Tensile strength	Grab strength
Puncture resistance	Puncture resistance
Tear resistance	Trapezoidal tear
Longevity:	
UV stability (if exposed)	UV resistance
Soil compatibility (where required)	Chemical; Biological
NOTE: 1. See Table 1-3 for specific test procedures.	

Holtz, and Bell (1986) needs additional work. The practical alternate is to use a wide strip tensile test (ASTM D 4595) as a measure of the in-soil strength. This point is discussed by Christopher and Holtz (1985) and Bonaparte, Holtz, and Giroud (1985). Traditional grab or narrow-strip tensile tests are **not** appropriate for obtaining design properties of reinforcing geosynthetics.

Therefore, strength and modulus are based on the ASTM D 4595 wide width tensile test. This test standard permits definition of tensile modulus in terms of: (i) initial tensile modulus; (ii) offset tensile modulus; or (iii) secant tensile modulus. Furthermore, the secant modulus may be defined between any two strain points. Geosynthetic modulus for design of embankments should be determined using a secant modulus, defined with the zero strain point and design strain limit (i.e., 2 to 10%) point.

The following minimum criteria for tensile strength of geosynthetics are recommended.

1. For ordinary cases, determine the design tensile strength T_d (the larger of T_g and T_{ls}) and the required secant modulus at 2 to 10% strain.
2. The ultimate tensile strength T_{ult} obviously must be greater that the design tensile strength, T_d. Note that T_g includes an inherent safety factor against overload and sudden failure that is equal to the rotational stability safety factor. The tensile strength requirements should

be increased to account for installation damage, depending on the severity of the conditions.

3. The strain of the reinforcement at failure should be at least 1.5 times the secant modulus strain to avoid brittle failure. For exceptionally soft foundations where the reinforcement will be subjected to very large tensile stresses during construction, the geosynthetic must have either sufficient strength to support the embankment itself, or the reinforcement and the embankment must be allowed to deform. In this case, an elongation at rupture of up to 50% may be acceptable. In either case, high tensile strength geosynthetics and special construction procedures (Section 7.8) are required.
4. If there is a possibility of tension cracks forming in the embankment or high strain levels occurring during construction (such as might occur, for example, with cohesive embankments), the lateral spreading strength, T_{ls}, at 2% strain should be required.
5. The required lateral spreading strength, T_{ls}, should be increased to account for creep and installation damage as the creep potential of the geosynthetic depends on the creep potential of the foundation. If significant creep is expected in the foundation, the creep potential of the geosynthetic at design stresses should be evaluated, recognizing that strength gains in the foundation will reduce the creep potential. Installation damage potential will depend on the severity of the conditions.
6. Strength requirements must be evaluated and specified for both the machine and cross machine directions of the geosynthetic. Usually, the seam strength controls the cross machine geosynthetic strength requirements.

Depending on the strength requirements, geosynthetic availability, and seam efficiency, more than one layer of reinforcement may be necessary to obtain the required tensile strength. If multiple layers are used, a granular layer of 200 to 300 mm must be placed between each successive geosynthetic layer or the layers must be mechanically connected (*e.g.*, sewn) together. Also, the geosynthetics must be strain compatible; that is, the same type of geosynthetic should be used for each layer.

For soil-geosynthetic friction values, either direct shear or pullout tests should be utilized. If test values are not available, Bell (1980) recommends that for sand embankments, the soil-geosynthetic friction is from $\frac{2}{3}\phi$ up to the full ϕ of the sand. Pullout tests by Holtz (1977) have shown that soil-geotextile friction is approximately equal to the ϕ of the sand. For clay soils, friction tests are definitely warranted and should be performed under all circumstances.

The creep properties of geosynthetics in reinforced soil systems are not well established. In-soil creep tests are possible but are far from routine today. For design, it is recommended that the working stress be kept much lower than the creep limit of the geosynthetic. Values of 40 to 60% of the ultimate stress are typically satisfactory for this purpose. A polyester will probably have less creep than a polypropylene or a polyethylene. Live loads versus dead loads also must be taken into account. Short-term live loadings are much less detrimental in terms of creep than sustained dead loads. And finally, as discussed in Section 7.3-3 Step 11, the relative rates of deformation of the geosynthetic versus the consolidation and strength gain of the foundation soil must be considered. In most cases, creep is not an issue in reinforced embankment stability.

7.4-2 Drainage Requirements

The geosynthetic must allow for free vertical drainage of the foundation soils to reduce pore pressure buildup below the embankment. Pertinent geosynthetic hydraulic properties are piping resistance and permeability (Table 7-1). It is recommended that the permeability of the geosynthetic be at least 10 times that of the underlying soil. The opening size should be selected based on the requirements of Section 2.3. The opening size should be a maximum to reduce the risk of clogging, while still providing retention of the underlying soil.

7.4-3 Environmental Considerations

For most embankment reinforcement situations, geosynthetics have a high resistance to chemical and biological attack; therefore, chemical and biological compatibility is usually not a concern. However, in unusual situations such as very low (*i.e.*, < 3) or very high (*i.e.*, > 9) pH soils, or other unusual chemical environments -- such as in industrial areas or near mine or other waste dumps -- the chemical compatibility of the polymer(s) in the geosynthetic should be checked to assure it will retain the design strength at least until the underlying subsoil is strong enough to support the structure without reinforcement.

7.4-4 Constructability (Survivability) Requirements

In addition to the design strength requirements, the geotextile or geogrid must also have sufficient strength to survive construction. If the geotextile is ripped, punctured, or torn during construction, support strength for the embankment structure will be reduced and failure could result. Constructability property requirements are listed in Table 7-1. (These are also called survivability requirements.) Tables 7-2 and 7-3 were developed by Haliburton, Lawmaster, and McGuffey (1982) specifically for reinforced embankment construction with varying subgrade conditions, construction equipment, and lift thicknesses (see also Christopher and Holtz, 1985). The specific property values in Table 7-4 are taken from the AASHTO M288 specification (1997), with very high and high relating to Class 1 and Class 2, respectively. Moderate and low categories of Table 7-4 relate to an M288 Class 3 geotextile. For all critical applications, high to very high survivability geotextiles and geogrids are recommended.

As the construction of the first lift of the embankment is analogous to construction of a temporary haul road, survivability requirements discussed in Section 5.9 are also appropriate here.

TABLE 7-2

REQUIRED DEGREE OF GEOSYNTHETIC SURVIVABILITY AS A FUNCTION OF SUBGRADE CONDITIONS AND CONSTRUCTION EQUIPMENT

SUBGRADE CONDITIONS	Construction Equipment and 150 to 300 mm Cover Material Initial Lift Thickness		
	Low Ground Pressure Equipment (≤ 30 kPa)	Medium Ground Pressure Equipment (> 30 kPa ≤ 60 kPa)	High Ground Pressure Equipment (>60 kPa)
Subgrade has been cleared of all obstacles except grass, weeds, leaves, and fine wood debris. Surface is smooth and level, and shallow depressions and humps do not exceed 150 mm in depth and height. All larger depressions are filled. Alternatively, a smooth working table may be placed.	Moderate/Low	Moderate	High
Subgrade has been cleared of obstacles larger than small- to moderate-sized tree limbs and rocks. Tree trunks and stumps should be removed or covered with a partial working table. Depressions and humps should not exceed 450 mm in depth and height. Larger depressions should be filled.	Moderate	High	Very High
Minimal site preparation is required. Trees may be felled, delimbed, and left in place. Stumps should be cut to project not more than 150 mm ± above subgrade. Geosynthetic may be draped directly over the tree trunks, stumps, large depressions and humps, holes, stream channels, and large boulders. Items should be removed only if, where placed, the Geosynthetic and cover material over them will distort the finished road surface.	High	Very High	Not Recommended

NOTES:
1. Recommendations are for 150 to 300 mm initial thickness. For other initial lift thickness:
   300 to 450 mm: Reduce survivability requirement one level
   450 to 600 mm: Reduce survivability requirement two levels
   > 600 mm: Reduce survivability requirement three levels
2. For special construction techniques such as prerutting, increase survivability requirement one level.
3. Placement of excessive initial cover material thickness may cause bearing failure of soft subgrades.
4. Note that equipment used for embankment construction (even *High Ground Pressure* equipment) have significantly lower ground contact pressures than equipment used for roadway construction (Table 5-2).

TABLE 7-3
REQUIRED DEGREE OF GEOSYNTHETIC SURVIVABILITY
AS A FUNCTION OF COVER MATERIAL AND CONSTRUCTION EQUIPMENT

CONSTRUCTION		COVER MATERIAL		
		Fine sand to +50 mm diameter gravel, rounded to subangular	Coarse aggregate with diameter up to one-half proposed lift thickness, may be angular	Some to most aggregate with diameter greater than one-half proposed lift thickness, angular and sharp-edged, few fines
150 to 300 mm Initial Lift Thickness	Low ground pressure equipment (≤30 kPa)	Moderate/Low	Moderate	High
	Medium ground pressure equipment (>30 kPa, ≤60 kPa)	Moderate	High	Very High
300 to 450 mm Initial Lift Thickness	Medium ground pressure equipment (>30 kPa, ≤60 kPa)	Moderate/Low	Moderate	High
	High ground pressure equipment (>60 kPa)	Moderate	High	Very High
450 to 600 mm Initial Lift Thickness	High ground pressure equipment (>60 kPa)	Moderate/Low	Moderate	High
> 600 mm Initial Lift Thickness	High ground pressure equipment (>60 kPa)	Moderate/Low	Moderate/Low	Moderate

NOTES:
1. For special construction techniques such as prerutting, increase geosynthetic survivability requirement one level.
2. Placement of excessive initial cover material thickness may cause bearing failure of soft subgrades.
3. Note that equipment used for embankment construction (even *High Ground Pressure* equipment) have significantly lower ground contact pressures than equipment used for roadway construction (Table 5-2).

TABLE 7-4
MINIMUM GEOTEXTILE PROPERTY REQUIREMENTS[1,2,3]
FOR GEOTEXTILE SURVIVABILITY (after AASHTO, 1997)

	ASTM Test Method	Units	Required Degree of Geotextile Survivability		
			Very High	High	Moderate / Low
Grab Strength	D 4632	N	1400	1100	800
Tear Strength	D 4533	N	500	400	300
Puncture Strength	D 4833	N	500	400	300
Burst Strength	D 3786	kPa	3500	2700	2100

NOTES: 1. Acceptance of geotextile material shall be based on ASTM D 4759.
2. Acceptance shall be based upon testing of either conformance samples obtained using Procedure A of ASTM D 4354, or based on manufacturer's certifications and testing of quality assurance samples obtained using Procedure B of ASTM D 4354.
3. Minimum; use value in weaker principal direction. All numerical values represent minimum average roll value (*i.e.*, test results from any sampled roll in a lot shall meet or exceed the minimum values in the table). Lot samples according to ASTM D 4354.

7.4-5 Stiffness and Workability

For extremely soft soil conditions, geosynthetic stiffness or workability may be an important consideration. The *workability* of a geosynthetic is its ability to support workpersons during initial placement and sewing operations and to support construction equipment during the first lift placement. Workability is generally related to geosynthetic stiffness; however, stiffness evaluation techniques and correlations with field workability are very poor (Tan, 1990). In the absence of any other stiffness information, ASTM Standard D 1388, Option A using a 50 x 300 mm specimen is recommended (see Christopher and Holtz, 1985). The values obtained should be compared with actual field performance to establish future design criteria. The workability guidelines based on subgrade CBR (Christopher and Holtz, 1985) are satisfactory for CBR > 1.0. For very soft subgrades, much stiffer geosynthetics are required. Other aspects of field workability such as water absorption and bulk density, should also be considered, especially on very soft sites.

7.4-6 Fill Considerations

When possible, the first few lifts of fill material just above the geosynthetic should be free-draining granular materials. This requirement provides the best frictional interaction between the geosynthetic and fill, as well as a drainage layer for excess pore water dissipation of the underlying soils. Other fill materials may be used above this layer as long as the strain compatibility of the geosynthetic is evaluated with respect to the backfill material, as discussed in Section 7.3-3, Step 8.

Most reinforcement analyses assume that the fill material is granular. In fact, in the past the use of cohesive soils together with geosynthetic reinforcement has been discouraged. This may be an unrealistic restriction, although there are problems with placing and compacting cohesive earth fills on especially soft subsoils. Furthermore, the frictional resistance between geosynthetics and cohesive soils is problematic. It may be possible to use composite embankments. Cohesionless fill could be used for the first 0.5 to 1 m; then the rest of the embankment could be constructed to grade with locally available materials.

7.5 DESIGN EXAMPLE

DEFINITION OF DESIGN EXAMPLE

- Project Description: A 4-lane highway is to be constructed over a peat bog. Alignment and anticipated settlement require construction of an embankment with an average height of 2 m. See project cross section figure.
- Type of Structure: embankment supporting a permanent paved road
- Type of Application: geosynthetic reinforcement
- Alternatives:
  i) excavate and replace - wetlands do not allow;
  ii) lightweight fill - high cost;
  iii) stone columns - soils too soft;
  iv) drainage and surcharge - yes; or
  v) very flat (8H:1V) slope - right-of-way restriction

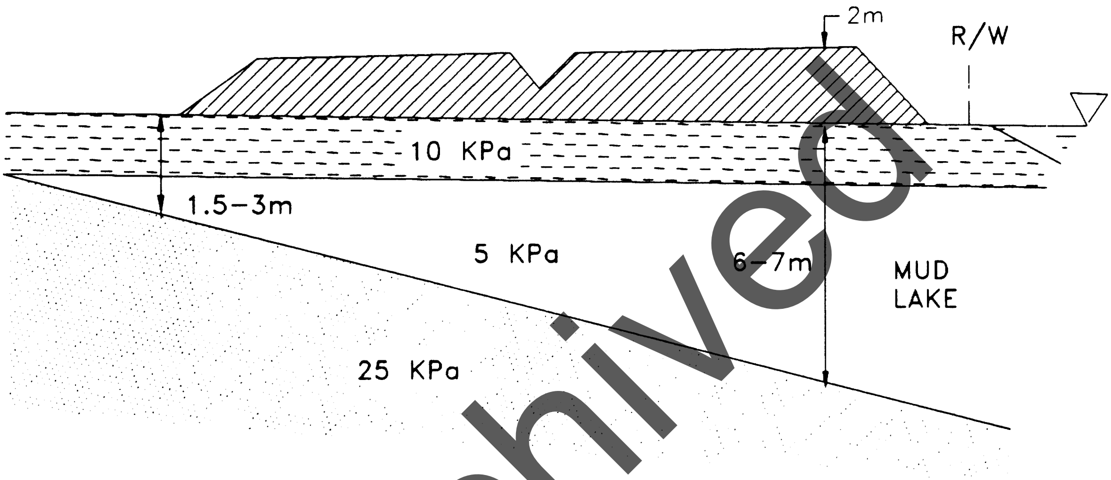
GIVEN DATA

- Geometry - as shown in project cross section figure
- Geosynthetic - geotextile (a geogrid also may be used for this example problem; however, this example represents an actual case history where a geotextile was used)
- Soils
  - subsurface exploration indicates $c_u = 5$ kPa in weakest areas
  - soft soils are underlain by firmer soils of $c_u = 25$ kPa
  - embankment fill soil will be sands and gravel
  - lightweight fill costs $250,000 more than sand/gravel
- Stability
  - Stability analyses of the unreinforced embankment were conducted with the STABL computer program. The most critical condition for embankments on soft soils is end-of-construction case; therefore, UU (unconsolidated, undrained) soil shear strength values are used in analyses.
  - Results of the analyses:
    a) With 4:1 side slopes and sand/gravel fill ($\gamma = 21.7$ kN/m^3), FS ≈ 0.72.

b) Since FS was substantially less than 1 for 4H:1V slopes, flatter slopes were evaluated, even though additional right-of-way would be required. With 8:1 side slopes and sand/gravel fill (γ = 21.7 kN/m^3), a FS ≈ 0.87 was computed.

c) Light-weight fill (γ = 15.7 kN/m^3) was also considered, with it, the FS varied between ≈0.90 to 1.15

- Transportation Department required safety factors are:
  Fs_{min} > 1.5 for *long-term* conditions
  FS_{allow} ≈ 1.3 for *short-term* conditions



Project Cross Section

REQUIRED

Design geotextile reinforcement to provide a stable embankment.

DEFINE

A. Geotextile function(s):

B. Geotextile properties required:

C. Geotextile specification:

SOLUTION

A. Geotextile function(s):
   Primary - reinforcement (for short-term conditions)
   Secondary - separation and filtration

B. Geotextile properties required:
   tensile characteristics
   interface shear strength
   survivability
   apparent opening size (AOS)

DESIGN

Design embankment with geotextile reinforcement to meet short-term stability requirements.

STEP 1. DEFINE DIMENSIONS AND LOADING CONDITIONS

See project cross section figure.

STEP 2. SUBSURFACE CONDITIONS AND PROPERTIES

Undrained shear strength provided in given data. Design for end-of-construction. Long-term design with drained shear strength parameters not covered within this example.

STEP 3. EMBANKMENT FILL PROPERTIES

sand and gravel, with
$\gamma_m = 21.7 \text{ kN/m}^3$ $\phi' = 35°$

STEP 4. ESTABLISH DESIGN REQUIREMENTS

- Transportation Department required safety factors are:
  $FS_{min} > 1.5$ for *long-term* conditions
  $FS_{min} \approx 1.3$ for *short-term* conditions

- settlement
  Primary consolidation must be completed prior to paving roadway.
  A total fill height of 2 m is anticipated to reach design elevation. **This height includes the additional fill material thickness to compensate for anticipated settlements.**

STEP 5. CHECK OVERALL BEARING CAPACITY

Recommended minimum safety factor (section 7.3-2) is 2.

A. Overall bearing capacity of soil, ignoring *footing* size is

$q_{ult} = c\ N_c$

$q_{ult} = 5\text{ kPa} \times 5.14 = 25.7\text{ kPa}$

Considering depth of embedment (*i.e.*, shearing will have to occur through the embankment for a bearing capacity failure) the bearing capacity is more accurately computed (see Meyerhof) as follows.

$N_c = 4.14 + 0.5\ B/D$ where, B = the base width of the embankment (~ 31 m), and
D = the average depth of the soft soil (~ 4.5 m)

$N_c = 4.14 + 0.5\ (31\text{ m} / 4.5\text{ m}) = 7.6$

$q_{ult} = 5\text{ kPa} \times 7.6 = 38\text{ kPa}$

maximum load, $P_{max} = \gamma_m\ H$

w/o a geotextile -

$P_{max} = 21.7\text{ kN/m}^3 \times 2\text{ m} = 43.4\text{ kPa}$

implies FS $= 38 \div 43.4 = 0.88$ $\therefore$ **NO GOOD**

with a geotextile, and assuming that the geotextile will result in an even distribution of the embankment load over the width of the geotextile (*i.e.*, account for the slopes at the embankment edges),

$P_{avg} = A_E\ \gamma_m / B$ where, A = cross section area of embankment, and
B = base width of the embankment

$P_{avg} = \{[½\ (31\text{ m} + 15\text{ m})\ 2\text{ m}]\ 21.7\text{ kN/m}^3\} / 31\text{ m}$

$P_{avg} = 32.2\text{ kPa} < q_{ult}$ worst case **Safety Factor Marginal**

Add berms to increase bearing capacity. Berms, 3 m wide, can be added within the existing right-of-way, increasing the base width to 37 m. With this increase in width,

$N_c = 4.14 + 0.5\ (37\text{ m} / 4.5\text{ m}) = 8.3$

$q_{ult} = 5\text{ kPa} \times 8.3 = 41.5\text{ kPa}$

and,

$P_{avg} = 32.2\text{ kPa}\ (31 / 37) = 27.0\text{ kPa}$

$FS = 41.5\text{ kPa} / 27.0\text{ kPa} = 1.54$ **Safety Factor O.K.**

B. Lateral squeeze

From FHWA Foundation Manual (Cheney and Chassie, 1993) -

If $\gamma_{fill} \times H_{fill} > 3c$, then lateral squeeze of the foundation soil can occur. Since $P_{max} = 43.4$ kPa is much greater than 3c, even considering the crust layer (c = 10 kPa), a rigorous lateral squeeze analysis was performed using the method by Jürgeson (1934). In this method, the lateral stress beneath the toe of the embankment is determined through charts or finite element analysis and compared to the shear strength of the soil. This method indicated a safety factor of approximately 1 for the 31 m base width. Adding the berm and extending the reinforcement to the toe of the berm decreases the potential for lateral squeeze as the lateral stress is reduced at the toe of the berm. The berms increased $FS_{SQUEEZZE}$ to greater than 1.5.

Also, comparing the reinforced design with Figure 7-5 indicates that the reinforced structure should be stable.

STEP 6. PERFORM ROTATIONAL SHEAR STABILITY ANALYSIS

Recommended minimum safety factor at end of construction (section 7.3-2) is 1.3.

The critical unreinforced failure surface is found through rotational stability methods. For this project, STABL4M was used and the critical, unreinforced surface FS = 0.72. As the soil supporting the embankment was highly compressible peat, the reinforcement was assumed to rotate such that $\beta = \theta$ (Figure 7-3 and Eq. 7-4b). Thus,

$$FS_{req} = \frac{M_R + T_g R}{M_D} \geq 1.3$$

$$T_g = \frac{1.3\, M_D - M_R}{R}$$

therefore, $T_g \approx 263$ kN/m

Feasible - yes. Geosynthetics are available which exceed this strength requirement, especially if multiple layers are used. For this project, an installation damage factor of approximately equal to 1.0, and 2 layers were used:

Bottom: 90 kN/m

Top: 180 kN/m

The use of 2 layers allowed the lower cost bottom material to be used over the full embankment plus berm width, while the higher strength and more expensive geotextile was only placed under the embankment section where it was required.

STEP 7. CHECK LATERAL SPREADING (SLIDING) STABILITY

Recommended minimum safety factor (section 7.3-2) is 1.5.

A. from Figure 7-4b:

$T = FS \times P_A = FS \times 0.5\, K_a\, \gamma_m\, H^2$

$T = 1.5\,(0.5)\,[\tan^2(45 - 35/2)]\,(21.7\ \text{kN/m}^3)\,(2\ \text{m})^2$

$T = 17.6$ kN/m

Use Reduction Factors (RF) = 3 for creep and 1 installation damage

therefore, $T_{ls} = 53$ kN/m

$T_{ls} < T_g$, therefore $T_{design} = T_g = 263$ kN/m

B. check sliding:

$$FS = \frac{b \tan \phi_{sg}}{K_a H}$$

$$FS = \frac{8m \times \tan 23}{0.27 \times 2m}$$

FS > 6, OK

STEP 8. ESTABLISH TOLERABLE DEFORMATION (LIMIT STRAIN) REQUIREMENTS

For cohesionless sand and gravel over deformable peat use ϵ = 10%

STEP 9. EVALUATE GEOSYNTHETIC STRENGTH REQUIRED IN LONGITUDINAL DIRECTION

From Step 7,
use $T_L = T_{ls} =$ 53 kN/m for reinforcement and seams in the cross machine (X-MD) direction

STEP 10. ESTABLISH GEOSYNTHETIC PROPERTIES

A. Design strength and elongation based upon ASTM D 4595
Ultimate tensile strength
$T_{d1} = T_{ult} \geq$ 90 kN/m in MD - Layer 1
$T_{d2} = T_{ult} \geq$ 180 kN/m in MD - Layer 2
$T_{ult} \geq$ 53 kN/m in X-MD - both layers
Reinforcement Modulus, J
$J = T_{ls} / 0.10 =$ 530 kN/m for limit strain of 10%
$J \geq$ 530 kN/m - MD and X-MD, both directions

B. seam strength
$T_{seam} \geq$ 53 kN/m with controlled fill placement

C. soil-geosynthetic adhesion
from testing, per ASTM D 5321, $\phi_{sg} \geq 23°$

D. geotextile stiffness based upon site conditions and experience

E. survivability and constructability requirements
Assume: 1. medium ground pressure equipment
2. 300 mm first lift
3. uncleared subgrade

Use a *Very High Survivability* geotextile (from Tables 7-2 and 7-3). Therefore, from Table 7-4, the geotextile reinforcement shall meet or exceed the minimum average roll values of:

Property	ASTM Test Method	Minimum Strength (N)
Grab Strength	D 4632	1400
Tear Resistance	D 4533	500
Puncture	D 4833	500
Burst	D 3786	3500

Drainage and filtration requirements -
Need grain size distribution of subgrade soils
Determine: maximum AOS for retention
minimum $k_g > k_s$
minimum AOS for clogging resistance

Complete Steps 11 through 15 to finish design.

STEP 11. PERFORM SETTLEMENT ANALYSIS

STEP 12. ESTABLISH CONSTRUCTION SEQUENCE REQUIREMENTS

STEP 13. ESTABLISH CONSTRUCTION OBSERVATION REQUIREMENTS

STEP 14. HOLD PRECONSTRUCTION MEETING

STEP 15. OBSERVE CONSTRUCTION

7.6 SPECIFICATIONS

Because the reinforcement requirements for soft-ground embankment construction will be project and site specific, standard specifications, which include suggested geosynthetic properties, are **not** appropriate, and special provisions or a separate project specification must be used. The following example includes most of the items that should be considered in a reinforced embankment project.

HIGH STRENGTH GEOTEXTILE FOR EMBANKMENT REINFORCEMENT
(from Washington Department of Transportation, November 1994)

Description
This work shall consist of furnishing and placing construction geotextile in accordance with the details shown in the plans.

Materials
Geotextile and Thread for Sewing
The material shall be a woven geotextile consisting only of long chain polymeric filaments or yarns formed into a stable network such that the filaments or yarns retain their position relative to each other during handling, placement, and design service life. At least 85 percent by weight of the of the material shall be polyolefins or polyesters. The material shall be free from defects or tears. The geotextile shall be free of any treatment or coating which might adversely alter its hydraulic or physical properties after installation. The geotextile shall conform to the properties as indicated in Table 1.

Thread used shall be high strength polypropylene, polyester, or Kevlar thread. Nylon threads will not be allowed.

Geotextile Properties

Table 1. Properties for high strength geotextile for embankment reinforcement.

Property	Test Method[1]	Geotextile Property Requirements[2]
AOS	ASTM D4751	.84 mm max. (#20 sieve)
Water Permittivity	ASTM D4491	0.02/sec. min.
Tensile Strength, min. in machine direction	ASTM D4595	(to be based on project specific design)
Tensile Strength, min. in x-machine direction	ASTM D4595	(to be based on project specific design)
Secant Modulus at 5% strain	ASTM D4595	(to be based on project specific design)
Seam Breaking Strength	ASTM D4884	(to be based on project specific design)
Puncture Resistance	ASTM D4833	330 N min.
Tear Strength, min. in machine and x-machine direction	ASTM D4533	330 N min.
Ultraviolet (UV) Radiation Stability	ASTM D4355	70% Strength Retained min., after 500 hrs in weatherometer

[1] The test procedures are essentially in conformance with the most recently approved ASTM geotextile test procedures, except geotextile sampling and specimen conditioning, which are in accordance with WSDOT Test Methods 914 an 915, respectively. Copies of these test methods are available at the Headquarters Materials Laboratory in Tumwater, Washington.
[2] All geotextile properties listed above are minimum average roll values (*i.e.*, the test result for any sampled roll in a lot shall meet or exceed the values listed).

Geotextile Approval

Source Approval

The Contractor shall submit to the Engineer the following information regarding each geotextile proposed for use:

Manufacturer's name and current address,
Full Product name,
Geotextile structure, including fiber/yarn type, and
Geotextile polymer type(s).

If the geotextile source has not been previously evaluated, a sample of each proposed geotextile shall be submitted to the Headquarters Materials Laboratory in Tumwater for evaluation. After the sample and required information for each geotextile type have arrived at the Headquarters Materials Laboratory in Tumwater, a maximum of 14 calendar days will be required for this testing. Source approval will be based on conformance to the applicable values from Table 1. Source approval shall not be the basis of acceptance of specific lots of material unless the lot sampled can be clearly identified, and the number of samples tested and approved meet the requirements of WSDOT Test Method 914.

Geotextile Samples for Source Approval

Each sample shall have minimum dimensions of 1.5 meters by the full roll width of the geotextile. A minimum of 6 square meters of geotextile shall be submitted to the Engineer for testing. The geotextile machine direction shall be marked clearly on each sample submitted for testing. The machine direction is defined as the direction perpendicular to the axis of the geotextile roll.

The geotextile samples shall be cut from the geotextile roll with scissors, sharp knife, or other suitable method which produces a smooth geotextile edge and does not cause geotextile ripping or tearing. The samples shall not be taken from the outer wrap of the geotextile nor the inner wrap of the core.

Acceptance Samples

Samples will be randomly taken by the Engineer at the jobsite to confirm that the geotextile meets the property values specified.

Approval will be based on testing of samples from each lot. A "lot" shall be defined for the purposes of this specification as all geotextile rolls within the consignment (i.e., all rolls sent to the project site) which were produced by the same manufacturer during a continuous period of production at the same manufacturing plant and have the same product name. After the samples and manufacturer's certificate of compliance have arrived at the Headquarters Materials Laboratory in Tumwater, a maximum of 14 calendar days will be required for this testing. If the results of the testing show that a geotextile lot, as defined, does not meet the properties required in Table 1, the roll or rolls which were sampled will be rejected. Two additional rolls for each roll tested which failed from the lot previously tested will then be selected at random by the Engineer for sampling and retesting. If the retesting shows that any of the additional rolls tested do not meet the required properties, the entire lot will be rejected. If the test results from all the rolls retested meet the required properties, the entire lot minus the roll(s) which failed will be accepted. All geotextile which has defects, deterioration, or damage, as determined by the Engineer, will also be rejected. All rejected geotextile shall be replaced at no expense to the Contracting Agency.

Certificate of Compliance

The Contractor shall provide a manufacturer's certificate of compliance to the Engineer which includes the following information about each geotextile roll to be used:

Manufacturer's name and current address,
Full product name,
Geotextile structure, including fiber/yarn type,
Geotextile polymer type(s),
Geotextile roll number, and
Certified test results.

Approval Of Seams

If the geotextile seams are to be sewn in the field, the Contractor shall provide a section of sewn seam before the geotextile is installed which can be sampled by the Engineer.

The seam sewn for sampling shall be sewn using the same equipment and procedures as will be used to sew the production seams. The seam sewn for sampling must be at least 2 meters in length. If the seams are sewn in the factory, the Engineer will obtain samples of the factory seam at random from any of the rolls to be used. The seam assembly description shall be submitted by the Contractor to the Engineer and will be included with the seam sample obtained for testing. This description shall include the seam type, stitch type, sewing thread type(s), and stitch density.

Construction Requirements

Geotextile Roll Identification, Storage, and Handling

Geotextile roll identification, storage, and handling of the geotextile shall be in conformance to ASTM D 4873. During periods of shipment and storage, the geotextile shall be kept dry at all times and shall be stored off the ground. Under no circumstances, either during shipment or storage, shall the materials be exposed to sunlight, or other form of light which contains ultraviolet rays, for more than five calendar days.

Preparation and Placement of the Geotextile Reinforcement

The area to be covered by the geotextile shall be graded to a smooth, uniform condition free from ruts, potholes, and protruding objects such as rocks or sticks. The Contractor may construct a working platform, up to 0.6 meters in thickness, in lieu of grading the existing ground surface. A working platform is required where stumps or other protruding objects which cannot be removed without excessively disturbing the subgrade are present. All stumps shall be cut flush with the ground surface and covered with at least 150 mm of fill before placement of the first geotextile layer. The geotextile shall be spread immediately ahead of the covering operation. The geotextile shall be laid with the machine direction perpendicular or parallel to centerline as shown in Plans. Perpendicular and parallel directions shall alternate. All seams shall be sewn. Seams to connect the geotextile strips end to end will not be allowed, as shown in the Plans. The geotextile shall not be left exposed to sunlight during installation for a total of more than 10 calendar days. The geotextile shall be laid smooth without excessive wrinkles. Under no circumstances shall the geotextile be dragged through mud or over sharp objects which could damage the geotextile. The cover material shall be placed on the geotextile in such a manner that a minimum of 200 mm of material will be between the equipment tires or tracks and the geotextile at all times. Construction vehicles shall be limited in size and weight such that rutting in the initial lift above the geotextile is not greater than 75 mm deep, to prevent overstressing the geotextile. Turning of vehicles on the first lift above the geotextile will not be permitted. Compaction of the first lift above the geotextile shall be limited to routing of placement and spreading equipment only. No vibratory compaction will be allowed on the first lift.

Small soil piles or the manufacturer's recommended method shall be used as needed to hold the geotextile in place until the specified cover material is placed.

Should the geotextile be torn or punctured or the sewn joints disturbed, as evidenced by visible geotextile damage, subgrade pumping, intrusion, or roadbed distortion, the backfill around the damaged or displaced area shall be removed and the damaged area repaired or replaced by the Contractor at no expense to the Contracting Agency. The repair shall consist of a patch of the same type of geotextile placed over the damaged area. The patch shall be sewn at all edges.

If geotextile seams are to be sewn in the field or at the factory, the seams shall consist of two parallel rows of stitching, or shall consist of a J-seam, Type SSn-1, using a single row of stitching. The two rows of stitching shall be 25 mm apart with a tolerance of plus or minus 13 mm and shall not cross, except for restitching. The stitching shall be a lock-type stitch. The minimum seam allowance, *i.e.*, the minimum distance from the geotextile edge to the stitch line nearest to that edge, shall be 40 mm if a flat or prayer seam, Type SSa-2, is used. The minimum seam allowance for all other seam types shall be 25 mm. The seam, stitch type, and the equipment used to perform the stitching shall be as recommended by the manufacturer of the geotextile and as approved by the Engineer.

The seams shall be sewn in such a manner that the seam can be inspected readily by the Engineer or his representative. The seam strength will be tested and shall meet the requirements stated in this Specification.

Embankment construction shall be kept symmetrical at all times to prevent localized bearing capacity failures beneath the embankment or lateral tipping or sliding of the embankment. Any fill placed directly on the geotextile shall be spread immediately. Stockpiling of fill on the geotextile will not be allowed.

The embankment shall be compacted using Method B of Section 2-03.3(14)C. Vibratory or sheepsfoot rollers shall not be used to compact the fill until at least 0.5 meters of fill is covering the bottom geotextile layer and until at least 0.3 meters of fill is covering each subsequent geotextile layer above the bottom layer.

The geotextile shall be pretensioned during installation using either Method 1 or Method 2 as described herein. The method selected will depend on whether or not a mudwave forms during placement of the first one or two lifts. If a mudwave forms as fill is pushed onto the first layer of geotextile, Method 1 shall be used. Method 1 shall continue to be used until the mudwave ceases to form as fill is placed and spread. Once mudwave formation ceases, Method 2 shall be used until the uppermost geotextile layer is covered with a minimum of 0.3 meters of fill. These special construction methods are not needed for fill construction above this level. If a mudwave does not form as fill is pushed onto the first layer of geotextile, then Method 2 shall be used initially and until the uppermost geotextile layer is covered with at least 0.3 meters of fill.

Method 1

After the working platform, if needed, has been constructed, the first layer of geotextile shall be laid in continuous transverse strips and the joints sewn together. The geotextile shall be stretched manually to ensure that no wrinkles are present in the geotextile. The fill shall be end-dumped and spread from the edge of the geotextile. The fill shall first be placed along the outside edges of the geotextile to form access roads. These access roads will serve three purposes: to lock the edges of the geotextile in place, to contain the mudwave, and to provide access as needed to place fill in the center of the embankment. These access roads shall be approximately 5 meters wide. The access roads at the edges of the geotextile shall have a minimum height of 0.6 meters when completed. Once the access roads are approximately 15 meters in length, fill shall be kept ahead of the filling operation, and the access roads shall be kept approximately 15 meters ahead of this filling operation as shown in the Plans. Keeping the mudwave ahead of this filling operation and keeping the edges of the geotextile from moving by use of the access roads will effectively pre-tension the geotextile. The geotextile shall be laid out no more than 6 meters ahead of the end of the access roads at any time to prevent overstressing of the geotextile seams.

Method 2

After the working platform, if needed, has been constructed, the first layer of geotextile shall be laid and sewn as in Method 1. The first lift of material shall be spread from the edge of the geotextile, keeping the center of the advancing fill lift ahead of the outside edges of the lift as shown in the Plans. The geotextile shall be manually pulled taut prior to fill placement. Embankment construction shall continue in this manner for subsequent lifts until the uppermost geotextile layer is completely covered with 0.3 meters of compacted fill.

Measurement

High strength geotextile for embankment reinforcement will be measured by the square meter for the ground surface area actually covered.

Payment

The unit contract price per square meter for "High Strength Geotextile For Embankment Reinforcement," shall be full pay to complete the work as specified.

7.7 COST CONSIDERATIONS

The cost analysis for a geosynthetic reinforced embankment includes:

1. Geosynthetic cost: including purchase price, factory prefabrication, and shipping.
2. Site preparation: including clearing and grubbing, and working table preparation.
3. Geosynthetic placement: related to field workability (see Christopher and Holtz, 1989),
   a) with no working table, or
   b) with a working table.
4. Fill material: including purchasing, hauling, dumping, compaction, allowance for additional fill due to embankment subsidence. (NOTE: Use free-draining granular fill for the lifts adjacent to geosynthetic to provide good adherence and drainage.)

7.8 CONSTRUCTION PROCEDURES

The construction procedures for reinforced embankments on soft foundations are extremely important. *Improper* fill placement procedures can lead to geosynthetic damage, nonuniform settlements, and even embankment failure. By the use of low ground pressure equipment, a properly selected geosynthetic, and proper procedures for placement of the fill, these problems can essentially be eliminated. Essential construction details are outlined below. The Washington State DOT Special Provision in Section 7.6 provides additional details.

A. Prepare subgrade:
   1. Cut trees and stumps flush with ground surface.
   2. Do not remove or disturb root or meadow mat.
   3. Leave small vegetative cover, such as grass and reeds, in place.
   4. For undulating sites or areas where there are many stumps and fallen trees, consider a working table for placement of the reinforcement. In this case, a lower strength sacrificial geosynthetic designed only for constructability can be used to construct and support the working table.

B. Geosynthetic placement procedures:
   1. Orient the geosynthetic with the machine direction perpendicular to the embankment alignment. No seams should be allowed parallel to the alignment. Therefore,
      - The geosynthetic rolls should be shipped in unseamed machine direction lengths equal to one or more multiples of the embankment design base width.
      - The geosynthetic should be manufactured with the largest machine width possible.
      - These widths should be factory-sewn to provide the largest width compatible with shipping and field handling.

   2. Unroll the geosynthetic as smoothly as possible transverse to the alignment. (Do not drag it.)

   3. Geotextiles should be sewn as required with all seams up and every stitch inspected. Geogrids should be positively joined by clamps, cables, pipes, etc.

4. The geosynthetic should be manually pulled taut to remove wrinkles. Weights (sand bags, tires, etc.) or pins may be required to prevent lifting by wind.

5. Before covering, the Engineer should examine the geosynthetic for holes, rips, tears, etc. Defects, if any, should be repaired by.
   - Large defects, should be replaced by cutting along the panel seam and sewing in a new panel.
   - Smaller defects, can be cut out and a new panel resewn into that section, if possible.
   - Defects less than 150 mm, can be overlapped a minimum of 1 m or more in all directions from the defective area. (Additional overlap may be required, depending on the geosynthetic-to-geosynthetic friction angle).

   NOTE: If a *weak link* exists in the geosynthetic, either through a defective seam or tear, the system will tell the engineer about it in a dramatic way -- spectacular failure! (Holtz, 1990)

C. Fill placement, spreading, and compaction procedures:

   1. Construction sequence for extremely soft foundations (when a mudwave forms) is shown in Figure 7-6.

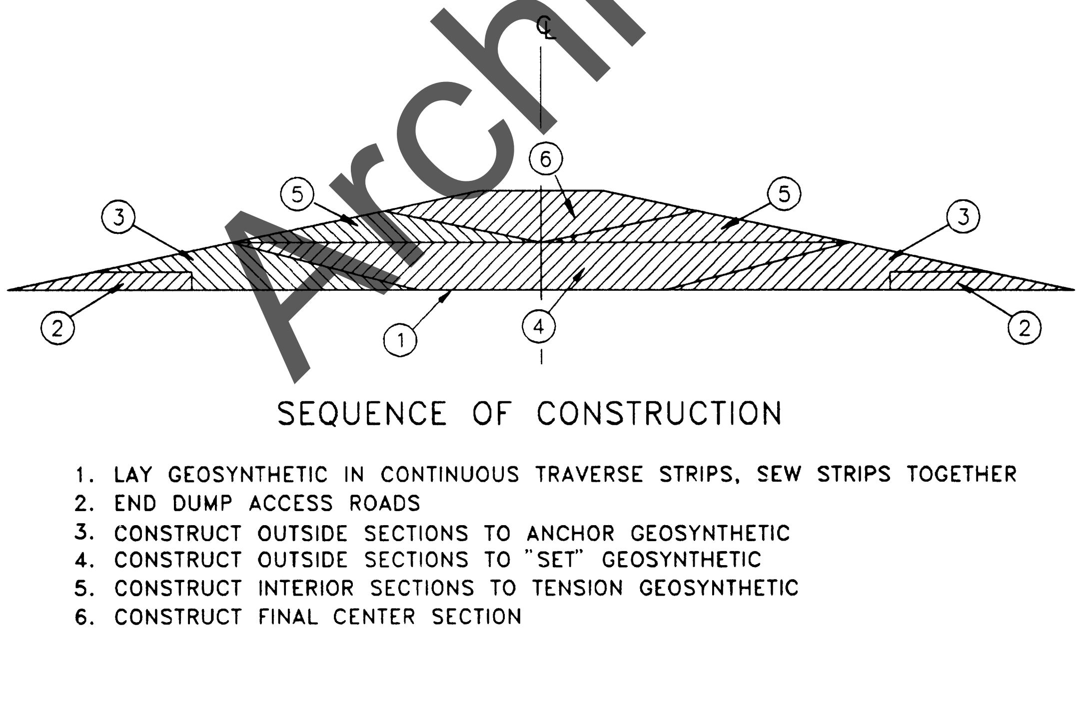


Figure 7-6 Construction sequence for geosynthetic reinforced embankments for extremely weak foundations (from Haliburton, Douglas and Fowler, 1977).

a. End-dump fill along edges of geosynthetic to form toe berms or access roads.
   - Use trucks and equipment compatible with constructability design assumptions (Table 7-1).
   - End-dump on the previously placed fill; do not dump directly on the geosynthetic.
   - Limit height of dumped piles, *e.g.*, to less than 1 m above the geosynthetic layer, to avoid a local bearing failure. Spread piles immediately to avoid local depressions.
   - Use lightweight dozers and/or front-end loaders to spread the fill.
   - Toe berms should extend one to two panel widths ahead of the remainder of the embankment fill placement.

b. After constructing the toe berms, spread fill in the area between the toe berms.
   - Placement should be parallel to the alignment and symmetrical from the toe berm inward toward the center to maintain a *U*-shaped leading edge (concave outward) to contain the mudwave (Figure 7-7).

c. Traffic on the first lift should be parallel to the embankment alignment; no turning of construction equipment should be allowed.
   - Construction vehicles should be limited in size and weight to limit initial lift rutting to 75 mm. If rut depths exceed 75 mm, decrease the construction vehicle size and/or weight.

d. The first lift should be compacted only by *tracking in place* with dozers or end-loaders.

e. Once the embankment is at least 600 mm above the original ground, subsequent lifts can be compacted with a smooth drum vibratory roller or other suitable compactor. If localized liquefied conditions occur, the vibrator should be turned off and the weight of the drum alone should be used for compaction. Other types of compaction equipment also can be used for nongranular fill.

2. After placement, the geosynthetic should be covered within 48 hours.

For less severe foundation conditions (*i.e.*, when no mudwave forms):

a. Place the geosynthetic with no wrinkles or folds; if necessary, manually pull it taut prior to fill placement.

b. Place fill symmetrically from the center outward in an inverted *U* (convex outward) construction process, as shown in Figure 7-8. Use fill placement to maintain tension in the geosynthetic.

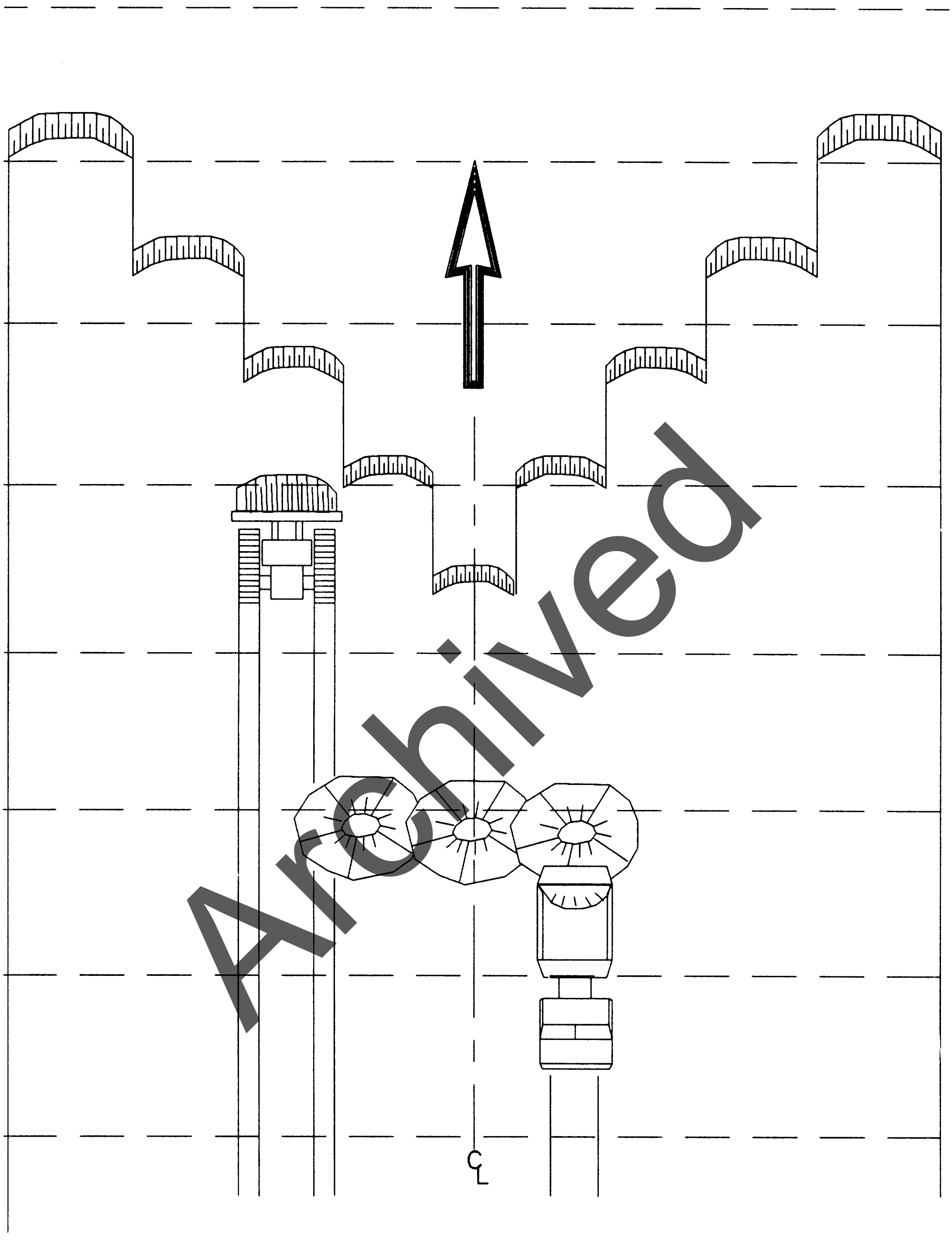


Figure 7-7 Placement of fill between toe berms on extremely soft foundations (CBR < 1) with a mud wave anticipated.

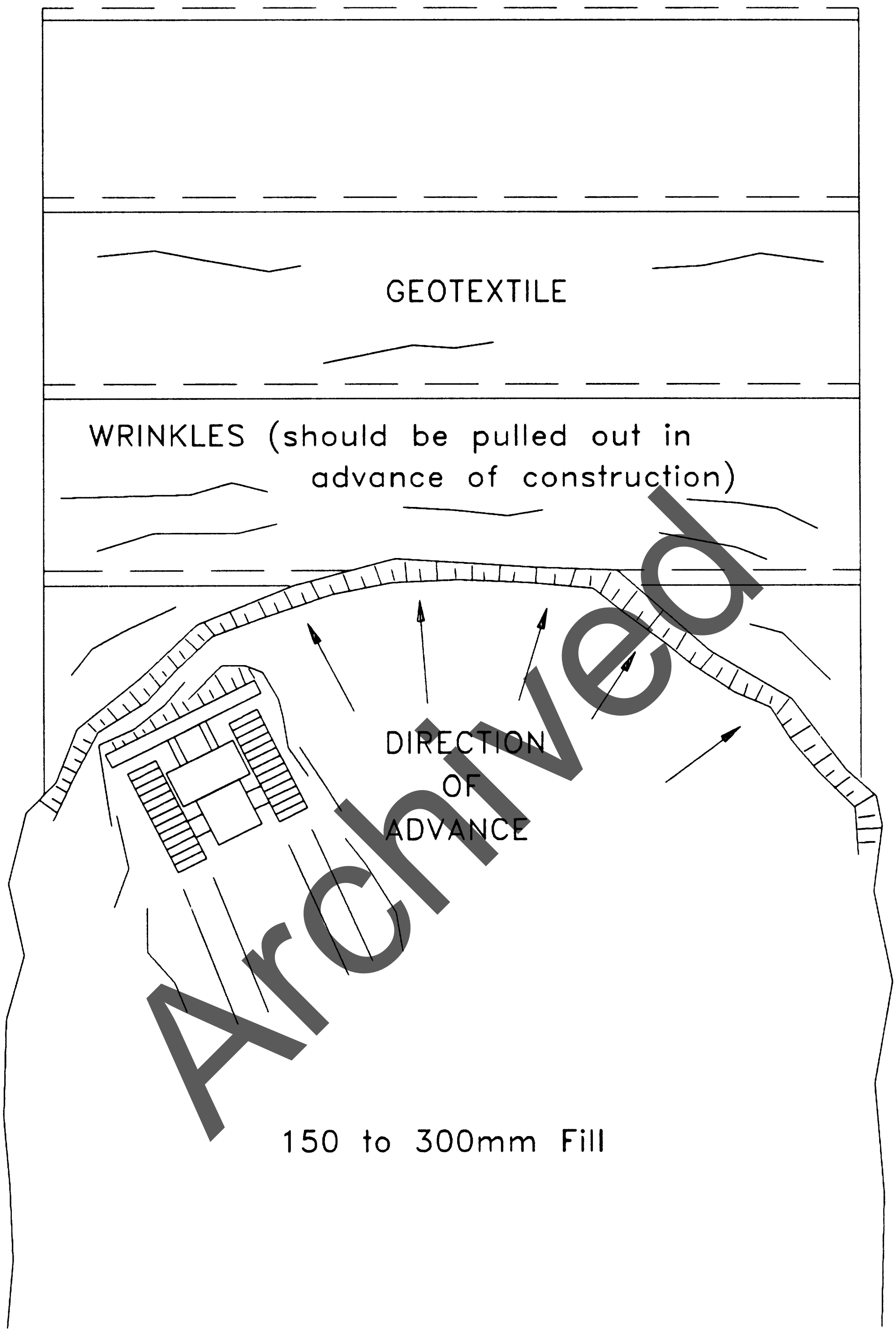


Figure 7-8 Fill placement to tension geotextile on moderate ground conditions; moderate subgrade (CBR > 1); no mud wave.

c. Minimize pile heights to avoid localized depressions.

d. Limit construction vehicle size and weight so initial lift rutting is no greater than 75 mm.

e. Smooth-drum or rubber-tired rollers may be considered for compaction of first lift; however, do not overcompact. If weaving or localized quick conditions are observed, the first lift should be compacted by tracking with construction equipment.

D. Construction monitoring:

1. Monitoring should include piezometers to indicate the magnitude of excess pore pressure developed during construction. If excessive pore pressures are observed, construction should be halted until the pressures drop to a predetermined safe value.

2. Settlement plates should be installed at the geosynthetic level to monitor settlement during construction and to adjust fill requirements appropriately.

3) Inclinometers should be considered at the embankment toes to monitor lateral displacement.

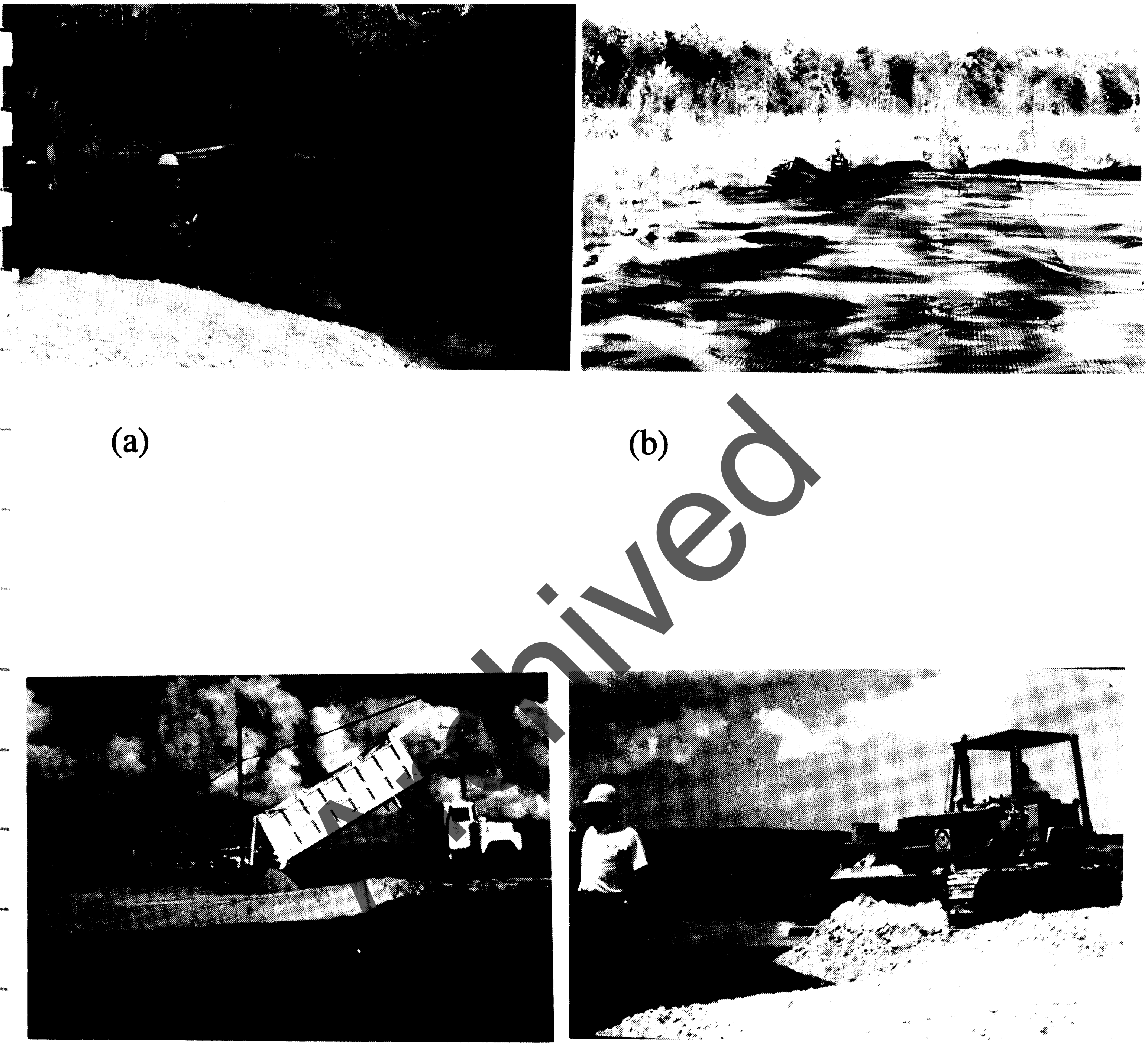
Photographs of reinforced embankment construction are shown in Figure 7-9.

7.9 INSPECTION

Since implemented construction procedures are crucial to the success of reinforced embankments on very soft foundations, competent and professional construction inspection is absolutely essential. Field personnel must be properly trained to observe every phase of the construction and to ensure that (1) the specified material is delivered to the project, (2) the geosynthetic is not damaged during construction, and (3) the specified sequence of construction operations are explicitly followed. Field personnel should review the checklist in Section 1.7.

7.10 REINFORCED EMBANKMENTS FOR ROADWAY WIDENING

Special considerations are required for widening of existing roadway embankments founded on soft foundations. Construction sequencing of fill placement, connection of the geosynthetic to the existing embankment, and settlements of both the existing and new fills must be addressed by the design engineer. Analytical techniques for geosynthetic reinforcement requirements are the same as those discussed in Section 7.3.



(a) (b) (c) (d)

Figure 7-9 Reinforced embankment construction; a) geosynthetic placement; b) geosynthetic placement; c) fill dumping; and d) fill spreading.

Two example roadway widening cross sections are illustrated in Figure 7-10. The addition of a vehicle lane on either side of an existing roadway (Fig. 7-10a) is feasible if the traffic can be detoured during construction. In this case, the reinforcement may be placed continuously across the existing embankment and beneath the two new outer fill sections. Placing both new lanes to one side of the embankment (Figs. 7-10b) may allow for maintaining one lane of traffic flow during construction. With the new fill placed to one side of the existing embankment, the anchorage of the geosynthetic into the existing embankment becomes an important design step.

Both the new fill sections and the existing fill sections will most likely settle during and after fill placement, although the amount of settlement will be greater for the new fill sections. The existing fills settle because of the influence of the new, adjacent fill loads on their foundation soils. The amount of settlements is a function of the foundation soils and amount of load (fill height). When fill is placed to one side of an embankment (Figs. 7-10b) the pavement may need substantial maintenance during construction and until settlements are nearly complete. Alternatively, light-weight fill could be used to reduce the settlement of the new fill and existing sections.

Note that the sections in Figure 7-10 do not indicate a geosynthetic reinforcement layer beneath the existing embankment section. Typically, the reinforcement for the embankment widening section would be designed assuming no contribution of existing section geosynthetic in reinforcing the new and combined sections. Therefore, connection of the new reinforcement to any existing reinforcement is normally not required.

For soft subgrades, where a mud wave is anticipated, construction should be parallel to the alignment with the outside fill placed in advance of the fill adjacent to the existing embankment. For firm subgrades, with no mudwave, fill may be placed outward, perpendicular to the alignment.

7.11 REINFORCEMENT OF EMBANKMENTS COVERING LARGE AREAS

Special considerations are required for constructing large reinforced areas, such as parking lots, toll plazas, storage yards for maintenance materials and equipment, and construction pads. Loads are more biaxial than conventional highway embankments, and design strengths and strain considerations must be the same in all directions. Analytical techniques for geosynthetic reinforcement requirements are the same as those discussed in Section 7.3. Because geosynthetic strength requirements will be the same in both directions, including across the seams, special seaming techniques must often be considered to meet required strength requirements. Ends of rolls may also require butt seaming. In this case, rolls of different lengths should be used to

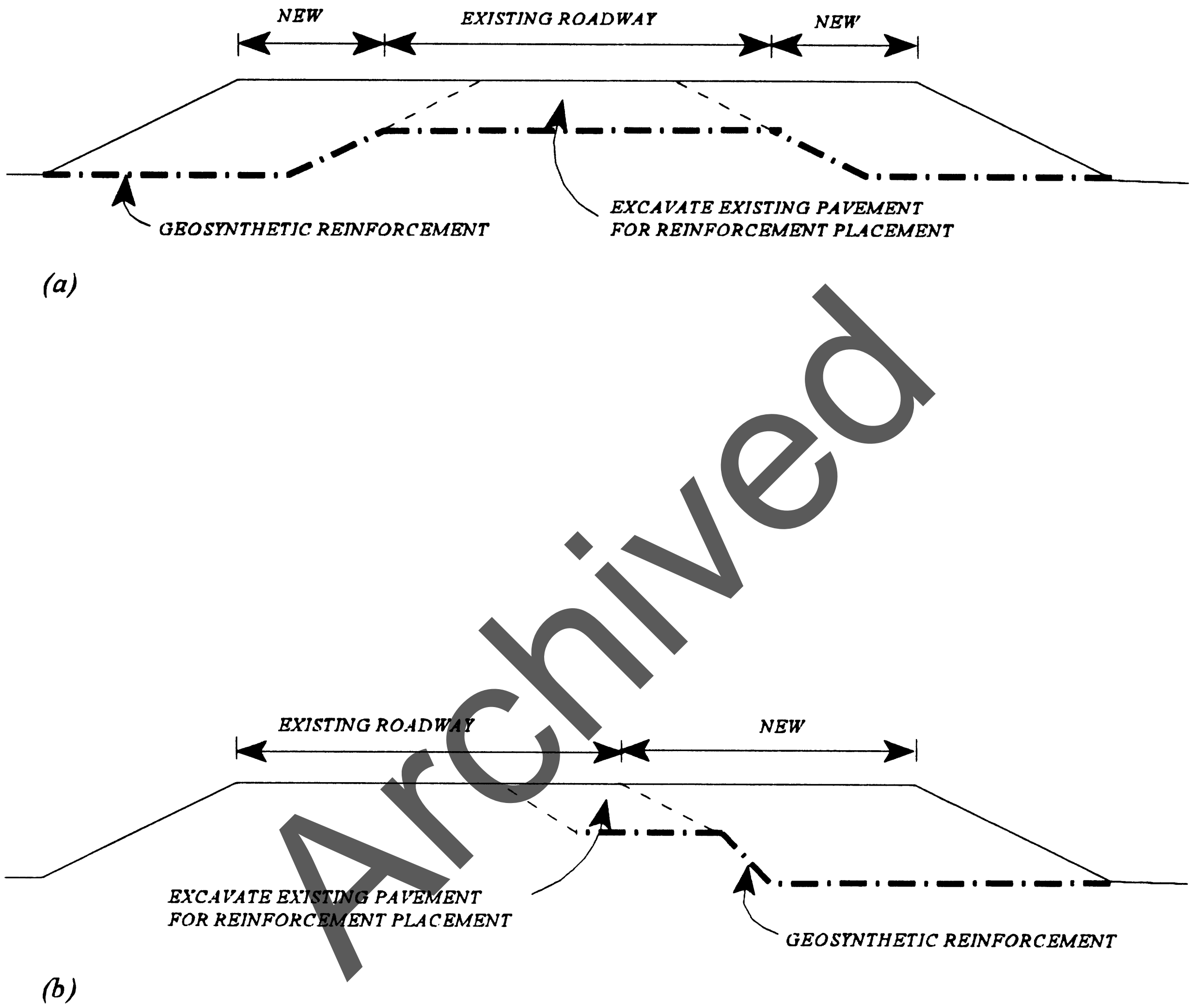


Figure 7-10 Reinforced embankment construction for roadway widening; a) fill placement on both sides of existing embankment; b) fill placement on one side of the existing fill.

stagger the butt seams. Two layers of fabric should be considered, with the bottom layer seams laid in one direction, and the top layer seams laid perpendicular to the bottom layer. The layers should be separated by a minimum lift thickness, usually 300 mm, soil layer.

For extremely soft subgrades, the construction sequence must be well planned to accommodate the formation and movement of mudwaves. Uncontained mudwaves moving outside of the construction can create stability problems at the edges of the embankment. It may be desirable to construct the fill in parallel embankment sections, then connect the embankments to cover the entire area. Another method staggers the embankment load by constructing a wide, low embankment with a higher embankment in the center. The outside low embankments are constructed first and act as berms for the center construction. Next, an adjacent low embankment is constructed from the outside into the existing embankment; then the central high embankment is spread over the internal adjacent low embankment. Other construction schemes can be considered depending on the specific design requirements. In all cases, a perimeter berm system is necessary to contain the mudwave.

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Key references are noted in bold type.

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8.0 REINFORCED SLOPES

8.1 BACKGROUND

Even if foundation conditions are satisfactory, slopes may be unstable at the desired slope angle. For new construction, the cost of fill, right-of-way, and other considerations may make a steeper slope desirable. Existing slopes, natural or manmade, may also be unstable, as is painfully obvious when they fail. As shown in Figure 8-1, multiple layers of geogrids or geotextiles may be placed in an earthfill slope during construction or reconstruction to reinforce the soil and provide increased slope stability. Reinforced soil slopes (RSS) are a form of mechanically stabilized earth which incorporate planar reinforcing elements in constructed earth-sloped structures with face inclinations less than 70° (Christopher, et al., 1990). MSE structures with face inclinations of 70° to 90° are classified as walls. These are addressed in Chapter 9.

In this chapter, analysis of the reinforcement and construction details required to provide a safe slope will be reviewed. The design method included in this chapter was first developed in the early 1980s for landslide repair in northern California. This approach has been validated by the thousands of reinforced soil slopes constructed over the last decade and through results of an extensive FHWA research program on reinforced soil structures as detailed in *Reinforced Soil Structures Volume I. Design and Construction Guidelines* and *Volume II. Summary of Research and Systems Information* (Christopher, et al., 1990). Contracting options and guideline specifications are included from *Guidelines for Design, Specification, and Contracting of Geosynthetic Mechanically Stabilized Earth Slopes on Firm Foundations* (Berg, 1993). The guidelines within chapter are consistent with those detailed in the recent FHWA Demonstration Project 82 manual *Mechanically Stabilized Earth Walls and Reinforced Soil Slopes, Design and Construction Guidelines* (Elias and Christopher, 1997).

8.2 APPLICATIONS

Geosynthetics are primarily used as slope reinforcement for construction of slopes to angles steeper than those constructed with the fill material being used, as illustrated in Figure 8-1a. Geosynthetics used in this manner can provide significant project economy by:

- creating usable land space at the crest or toe of the reinforced slope;
- reducing the volume of fill required;
- allowing the use of less-than-high-quality fill; and
- eliminating the expense of facing elements required on MSE walls.

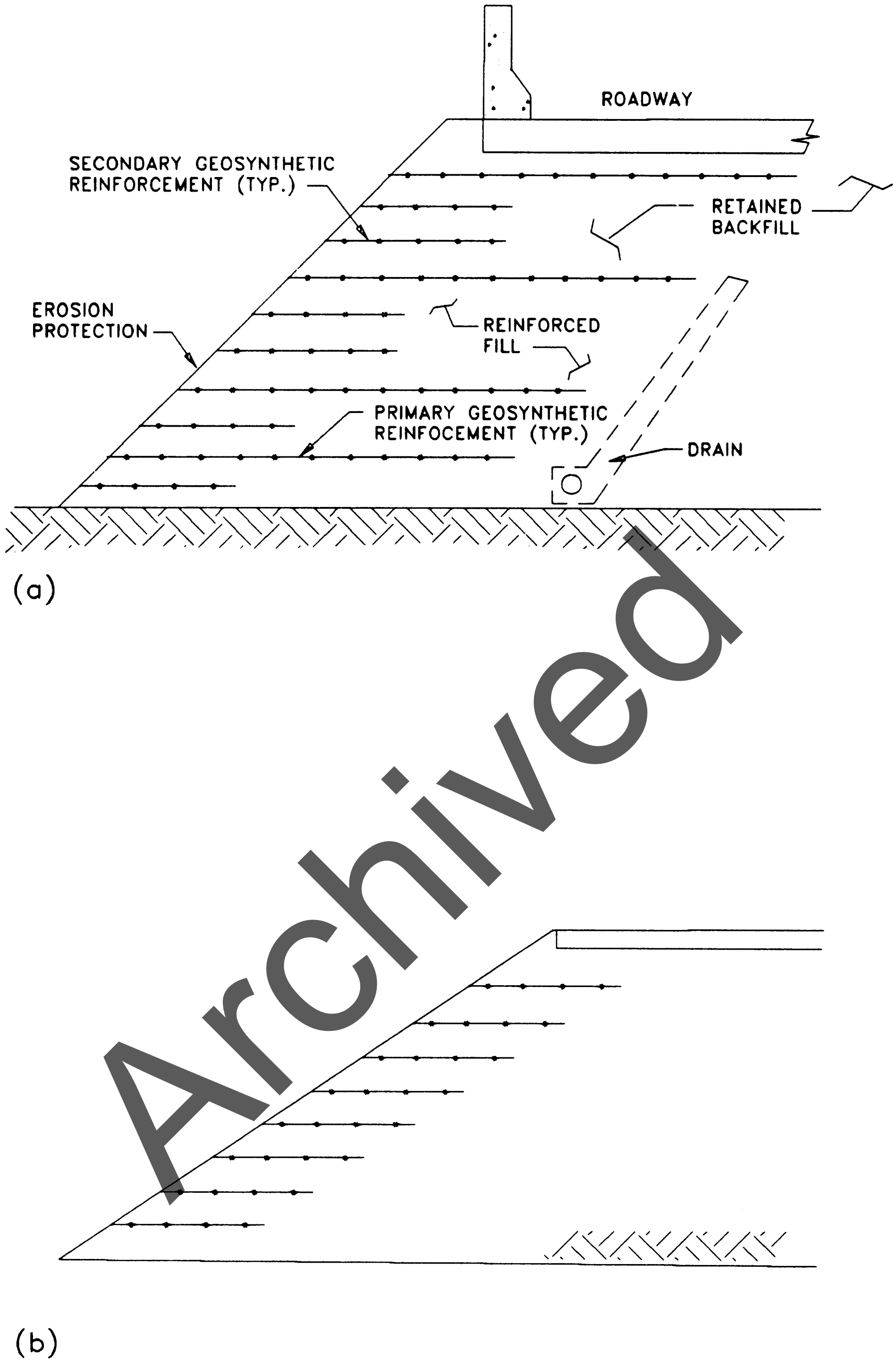


Figure 8-1 Use of geosynthetics in engineered slopes: (a) to increase stability of a slope; and (b) to provide improved compaction and surficial stability at edge of slopes (after Berg, et al., 1990).

Applications which highlight some of these advantages, illustrated in Figure 8-2, include:

- construction of new highway embankments;
- construction of alternatives to retaining walls;
- widening of existing highway embankments; and
- repair of failed slopes.

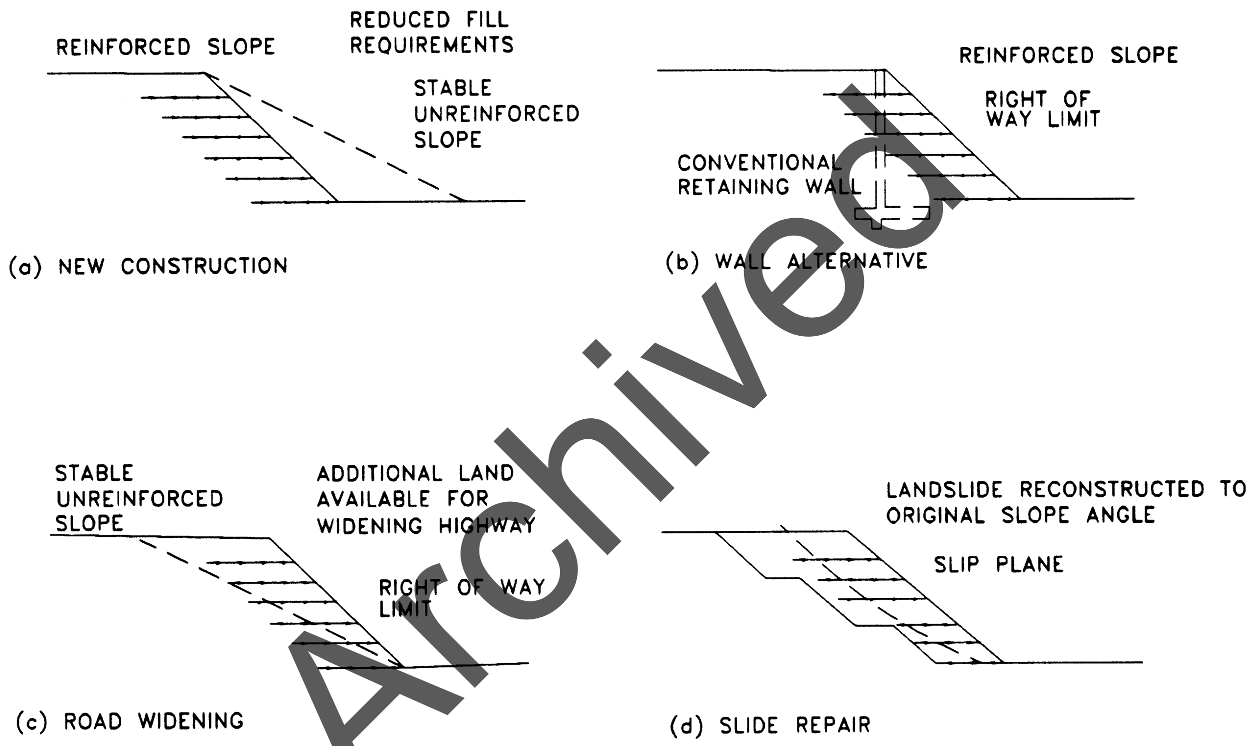


Figure 8-2 Applications of RSSs: (a) construction of new embankments; (b) alternative to retaining walls; (c) widening existing embankments; and (d) repair of landslides (after Tensar, 1987).

The design of reinforcement for safe, steep slopes requires rigorous analysis. The design of reinforcement for these applications is critical because reinforcement failure results in slope failure. To date, several thousand reinforced slope structures have been successfully constructed at various slope face angles. The tallest structure constructed in the U.S. to date, is a 1H:1V reinforced slope 33.5 m high (Bonaparte, et al., 1989).

A second purpose of geosynthetics placed at the edges of a compacted fill slope is to provide lateral resistance during compaction (Iwasaki and Watanabe, 1978) and surficial stability (Thielen and Collin, 1993). The increased lateral resistance allows for increased compacted soil density over that normally achieved and provides increased lateral confinement for soil at the face. Even modest amounts of reinforcement in compacted slopes have been found to prevent sloughing and reduce slope erosion. Edge reinforcement also allows compaction equipment to operate safely near the edge of the slope. Further compaction improvements have been found in cohesive soils using geosynthetics with in-plane drainage capabilities (*e.g.*, nonwoven geotextiles) which allow for rapid pore pressure dissipation in the compacted soil (Zornberg and Mitchell, 1992).

Design for the compaction improvement application is simple. Place a geogrid or geotextile that will survive construction at every lift or every other lift in a continuous plane along the edge of the slope. Only narrow strips, about 1.2 to 2 m in width, at 0.3 to 0.5 m vertical spacing are required. No reinforcement design is required if the overall slope is found to be safe without reinforcement. Where the slope angle approaches the angle of repose of the soil, a face stability analysis should be performed using the method presented in Section 8.3. Where reinforcement is required by analysis, the narrow strip geosynthetic may be considered as a secondary reinforcement used to improve compaction and to stabilize the slope face between primary layers.

Other applications of reinforced soil slopes include:

- upstream/downstream face stability and increased height of dams;
- construction of permanent levees and temporary flood control structures;
- steepening abutments and decreasing bridge spans;
- temporary road widening for detours; and
- embankment construction with wet, fine-grained soils.

8.3 DESIGN GUIDELINES FOR REINFORCED SLOPES

8.3-1 Design Concepts

The overall design requirements for reinforced slopes are similar to those for unreinforced slopes: the factor of safety must be adequate for both the short-term and long-term conditions and for all possible modes of failure.

Permanent, critical reinforced structures should be designed using comprehensive slope stability analyses. A structure may be considered permanent if its design life is greater than 3 years. An application is considered critical if there is mobilized tension in the reinforcement for the life of the structure, if reinforcement failure results in failure of the structure, or if the consequences of failure include personal injury or significant property damage (Bonaparte and Berg, 1987). A RSS is typically not considered critical if the safety factor against instability of the same unreinforced slope is greater than 1.1, and the reinforcement is used to increase the safety factor.

Failure modes of reinforced slopes (Berg, et al., 1989) include:

1. internal, where the failure plane passes through the reinforcing elements;
2. external, where the failure surface passes behind and underneath the reinforced mass; and
3. compound, where the failure surface passes behind and through the reinforced soil mass.

In many cases, the stability safety factor will be approximately equal in two or all three modes.

Reinforced slopes are currently analyzed using modified versions of the classical limit equilibrium slope stability methods. A circular or wedge-type potential failure surface is assumed, and the relationship between driving and resisting forces or moments determines the slope's factor of safety. Based on their tensile capacity and orientation, reinforcement layers intersecting the potential failure surface are assumed to increase the resisting moment or force. The tensile capacity of a reinforcement layer is the minimum of its allowable pullout resistance behind, or in front of, the potential failure surface and/or its long-term design tensile strength, whichever is smaller. A wide variety of potential failure surfaces must be considered, including deep-seated surfaces through or behind the reinforced zone. The slope stability factor of safety is taken from the critical surface requiring the maximum reinforcement. Detailed design of reinforced slopes is performed by determining the factor of safety with sequentially modified reinforcement layouts until the target factor of safety is achieved.

Ideally, reinforced slope design is accomplished using a conventional slope stability computer program modified to account for the stabilizing effect of reinforcement. Such programs should account for reinforcement strength and pullout capacity, compute reinforced and unreinforced safety factors automatically, and have a searching routine to help locate critical surfaces.

Several reinforced slope programs are commercially available, though some are limited to specific soil and reinforcement conditions. These programs generally do not design the reinforcement but allow for an evaluation of a given reinforcement layout. An iterative approach then follows to optimize either the reinforcement or the layout. Some of these programs are limited to simple soil profiles and, in some cases, reinforcement layouts. Vendor-supplied programs are, in many cases, reinforcement specific. These programs could be used to provide a preliminary evaluation or to check a detailed analysis.

A generic program, Reinforced Soil Slopes (RSS), for both reinforcement design and evaluation of almost any condition, has recently been developed by the FHWA. The program is based on the design method presented in Section 8.3-2, Design of Reinforced Slopes, Steps 5 and 6; Christopher, et al. (1990); and as summarized in FHWA Demonstration Project 82 manual (Elias and Christopher, 1997).

8.3-2 Design of Reinforced Slopes

The steps for design of a reinforced soil slope are:

STEP 1. Establish the geometric, loading, and performance requirements for design.

STEP 2. Determine the subsurface stratigraphy and the engineering properties of the in situ soils.

STEP 3. Determine the engineering properties of the available fill soils.

STEP 4. Evaluate design parameters for the reinforcement (design reinforcement strength, durability criteria, soil-reinforcement interaction).

STEP 5. Determine the factor of safety of the unreinforced slope.

STEP 6. Design reinforcement to provide stable slope.
Method A - Direct reinforcement design
Method B - Trial reinforcement layout analysis

STEP 7. Check external stability.

STEP 8. Evaluate requirements for subsurface and surface water control.

Details required for each step, along with equations for analysis, are presented in section 8.3-3. The procedure in section 8.3-3 assumes that the slope will be constructed on a stable foundation (*i.e.*, a circular or wedge-shaped failure surface through the foundation is not critical and local bearing support is clearly adequate). It does not include recommendations for deep-seated failure analysis. The user is referred to Chapter 7 for use of reinforcement in embankments over weak foundation soils.

For slide repair applications, it is also very important that solutions address the cause of original failure. Make sure that the new reinforced soil slope will not have the same problems. If water table or erratic water flows exist, particular attention must be paid to drainage. In natural soils, it is also necessary to identify any weak seams that could affect stability.

8.3-3 Reinforced Slope Design Guidelines

The following provides design procedure details for reinforced soil slopes.

STEP 1. Establish the geometric, load, and performance requirements for design (Figure 8-3).

Geometric and load requirements:

a. Slope height, H.
b. Slope angle, β.
c. External (surcharge) loads:
   - Surcharge load, q
   - Temporary live load, Δq
   - Design Seismic acceleration, A_m (see Division 1A, AASHTO *Standard Specifications for Highway Bridges* (1996))
   - Traffic barrier load — see article 2.7 of 1992 AASHTO *Standard Specifications for Highway Bridges* and AASHTO 1989 Roadside Design Guide

Performance requirements:

a. External stability and settlement
   - Horizontal sliding of the MSE mass along its base, FS $\geq$ 1.3
   - External, deep-seated, FS $\geq$ 1.3
   - Local bearing failure (lateral squeeze), FS $\geq$ 1.3
   - Dynamic loading: FS $\geq$ 1.1.
   - Settlement — post construction magnitude and time rate based on project requirements.

b. Compound failure modes (for planes passing behind and through the reinforced mass)
   - Compound failure surfaces, FS $\geq$ 1.3

c. Internal stability
   - Internal failure surfaces, FS $\geq$ 1.3

STEP 2. Determine the engineering properties of the in situ soils in the slope.

a. Determine the foundation and retained soil (*i.e.*, soil beneath and behind reinforced zone) profiles along the alignment (every 30 to 60 m, depending on the homogeneity of the subsurface profile) deep enough to evaluate a potential deep-seated failure (recommended exploration depth is twice the height of the slope or to refusal).
b. Determine the foundation and retained fill soil strength parameters (c_u, ϕ_u or c' and ϕ'); unit weight (wet and dry); and consolidation parameters (C_c, C_r, c_v and σ'_p) for each layer.
c. Locate the groundwater table, d_w, and piezometric surfaces (especially important if water will exit slope).
d. For slope and landslide repairs, identify the cause of instability and locate the previous failure surface.

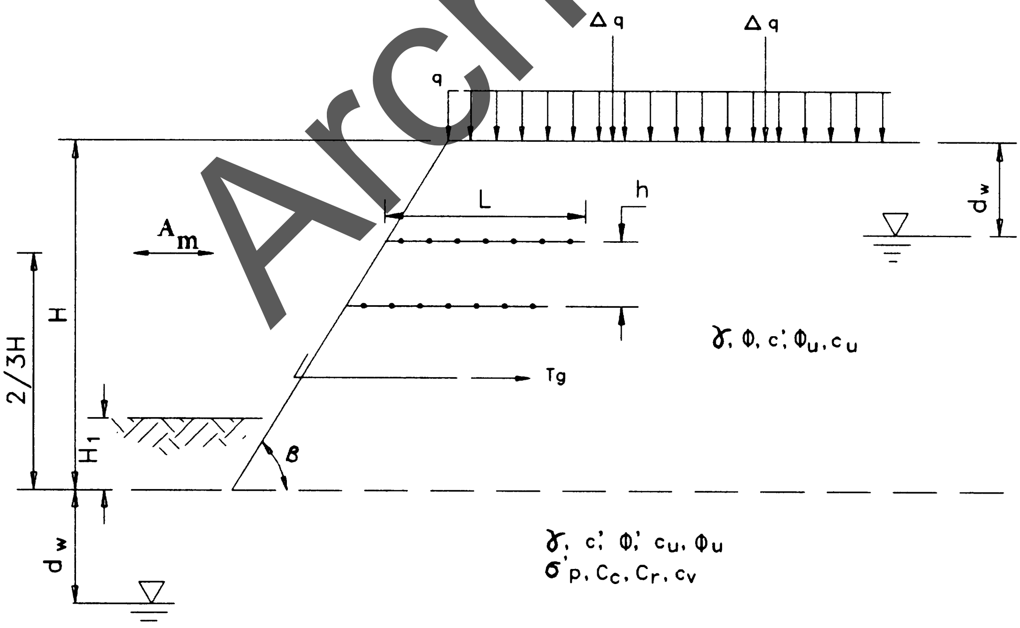


Figure 8-3 Requirements for design of a reinforced slope.

STEP 3. Determine properties of reinforced fill and, if different, the fill behind the reinforced zone.

See recommendations in Section 8.4-1.

a. Gradation and plasticity index
b. Compaction characteristics and placement requirements
c. Shear strength parameters, c_u, ϕ_u or c', ϕ'
d. Chemical composition of soil; pH

STEP 4. Evaluate design parameters for the reinforcement.

See recommendations in section 8.4-1 and Appendix K.

a. Allowable geosynthetic strength, T_{al} = ultimate strength (T_{Ult}) ÷ reduction factors (RF) for creep, installation damage, and durability
b. Pullout Resistance: Use FS $\geq$ 1.5 for granular soils and FS $\geq$ 2 for cohesive soils, with a minimum embedment length, L_e = 1 m.

STEP 5. Check unreinforced stability.

a. Evaluate unreinforced stability to determine: if reinforcement is required; critical nature of the design (*i.e.*, unreinforced FS $\leq$ or $\geq$ 1); potential deep-seated failure problems; and the extent of the reinforced zone.
   - Perform a stability analysis using conventional stability methods (see FHWA *Soils and Foundations Workshop Manual*, 1993) to determine safety factors and driving moments for potential failure surfaces.
   - Use both circular arc and sliding wedge methods, and consider failure through the toe, through the face (at several elevations), and deep seated below the toe. Failure surface exit points should be defined within each of the potential failure zones.

A number of stability analysis computer programs are available for rapid evaluation (*e.g.*, the STABL family of programs developed at Purdue University including the current version, STABL5M, and the program XSTABL developed at the University of Idaho). In all cases, you should perform a few calculations by hand to verify reasonability of the computer program.

b. Determine the size of the critical zone to be reinforced.

- Examine the full range of potential failure surfaces with safety factors less than or equal to the slope's target safety.
- Plot all of these surfaces on the slope's cross-section.
- The surfaces that just meet the target factor of safety roughly envelope the limits of the critical zone to be reinforced, as illustrated in Figure 8-4.

c. Critical failure surfaces extending below the toe of the slope indicate deep foundation and edge bearing capacity problems that must be addressed prior to design completion. For such cases, a more-extensive foundation analysis is warranted. Geosynthetics may be used to reinforce the base of the embankment and to construct toe berms for improved embankment stability, as reviewed in Chapter 7. Other foundation improvement measures should be considered.

STEP 6. Design reinforcement to provide for a stable slope.

Several approaches are available for the design of slope reinforcement, many of which are contained in Christopher and Holtz (1985, see Chapter 5 and Appendix D). Two methods are presented in this section. The first method uses a direct design approach to obtain the reinforcing requirements. The second method, analyzes trial reinforcement layouts. The computer program RSS developed by the FHWA incorporates both approaches.

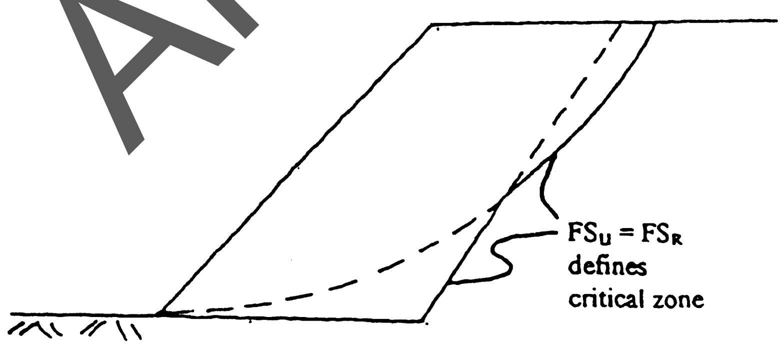


Figure 8-4 Critical zone defined by rotational and sliding surface that meet the required safety factor.

Method A - Direct design approach.

The first method, presented in Figure 8-5 for a rotational slip surface, uses any conventional slope stability computer program, and the steps necessary to manually calculate the reinforcement requirements. This design approach can accommodate fairly complex conditions depending on the analytical method used (*e.g.*, Bishop, Janbu, etc.).

The assumed orientation of the reinforcement tensile force influences the calculated slope safety factor. In a conservative approach, the deformability of the reinforcements is not taken into account; therefore, the tensile forces per unit width of reinforcement, T_r, are always assumed to be horizontal to the reinforcements, as illustrated in Figure 8-5. However, close to failure, the reinforcements may elongate along the failure surface, and an inclination from the horizontal can be considered. Tensile force direction is therefore dependent on the extensibility of the reinforcements used, and for continuous extensible geosynthetic reinforcement, a T inclination tangent to the sliding surface is recommended. For discontinuous strips of geosynthetic reinforcement, a horizontal orientation should be conservatively assumed.

Judgment and experience in selecting of the most appropriate design is required.
The following design steps are necessary:

a. Calculate the total reinforcement tension per unit width of slope, T_S, needed to obtain the required factor of safety for each potential failure circle inside the critical zone (Step 5) that extends through or below the toe of the slope (see Figure 8-5). Use the following equation:

$$T_S = (FS_R - FS_U)\ \frac{M_D}{D}$$

where:

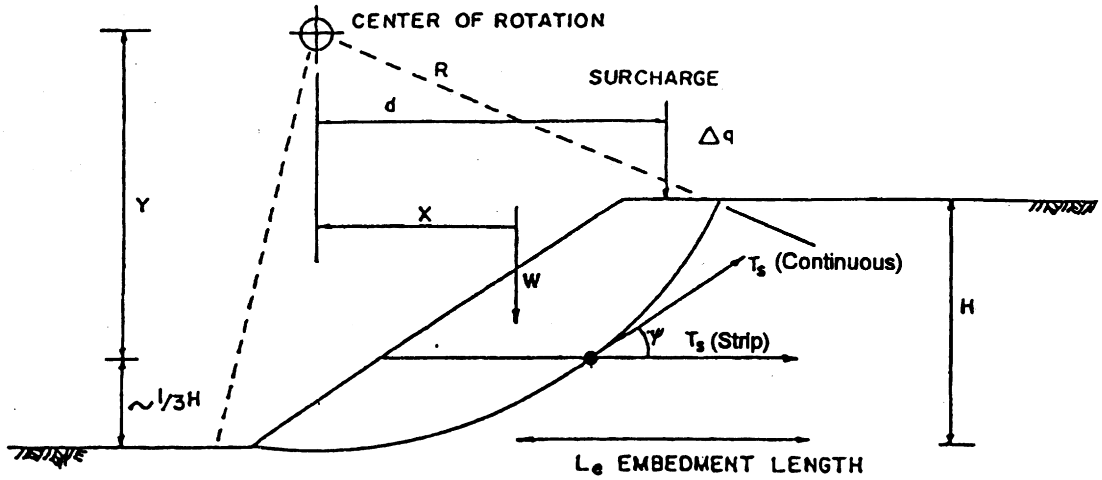
T_S = sum of required tensile force per unit width of reinforcement (considering rupture and pullout) in all reinforcement layers intersecting the failure surface;

M_D = driving moment about the center of the failure circle;

D = the moment arm of T_S about the center of failure circle,
= radius of circle R for **continuous, sheet type geosynthetic reinforcement** (*i.e.*, assumed to act tangentially to the circle).

FS_R = target minimum slope safety factor which is applied to both the soil and reinforcement; and

FS_U = unreinforced slope safety factor.



Factor of safety of unreinforced slope:

$$FS_u = \frac{Resisting\ Moment\ (M_R)}{Driving\ Moment\ (M_D)} = \frac{L_{SP}\ \tau_f\ R}{Wx + qd}$$

where: W = weight of sliding earth mass
L_{SP} = length of slip plane
q = surcharge
τ_f = shear strength of soil

Factor of safety of reinforced slope:

$$FS = FS_u + \frac{T_s\ D}{M_D}$$

where: T_s = sum of available tensile force per width of reinforcement for all reinforcement layers
D = moment arm of T_s about the center of rotation
= R for continuous geosynthetic reinforcement
= for discontinuous, strip type geosynthetic reinforcement

Figure 8-5 Rotational shear approach to determine required strength of reinforcement.

- $T_{S\text{-}MAX}$, the largest T_S calculated establishes the total required design tension.
- **NOTE: The minimum safety factor usually does not control the location of $T_{S\text{-}MAX}$, the most critical surface is the surface requiring the largest magnitude of reinforcement.**

b. As a check of computer-generated results, determine the total required design tension per unit width of slope, $T_{S\text{-}MAX}$, using the charts in Figure 8-6. Compare results with Step 6a. If substantially different, check the validity of the charts based on the limiting assumptions listed in the figure and recheck Steps 5 and 6a.

Figure 8-6 provides a method for quickly checking the computer-generated results. The charts are based upon simplified analysis methods of two-part and one-part wedge-type failure surfaces and are limited by the assumptions noted on the figure. Note that Figure 8-6 is **not** intended to be a single design tool. Other design charts are also available from Jewell (1984); Jewell (1990); Werner and Resl (1986); Ruegger (1986); and Leshchinsky and Boedeker (1989). Several computer programs are also available for analyzing a slope with given reinforcement and can also be used as a check.

c. Determine the distribution of reinforcement:

- For low slopes ($H \leq 6$ m) assume a uniform distribution of reinforcement and use $T_{S\text{-}MAX}$ to determine spacing or reinforcement tension requirements in Step 6d.

- For high slopes ($H > 6$ m), divide the slope into two (top and bottom) or three (top, middle, and bottom) reinforcement zones of equal height and use a factored $T_{S\text{-}MAX}$ in each zone for spacing or reinforcement requirements in Step 6d. The total required tension in each zone is found from the following equations:

For two zones:

$$T_{Bottom} = \tfrac{3}{4}\, T_{S\text{-}MAX}$$
$$T_{Top} = \tfrac{1}{4}\, T_{S\text{-}MAX}$$

For three zones:

$$T_{Bottom} = \tfrac{1}{2}\, T_{S\text{-}MAX}$$
$$T_{Middle} = \tfrac{1}{3}\, T_{S\text{-}MAX}$$
$$T_{Top} = \tfrac{1}{6}\, T_{S\text{-}MAX}$$

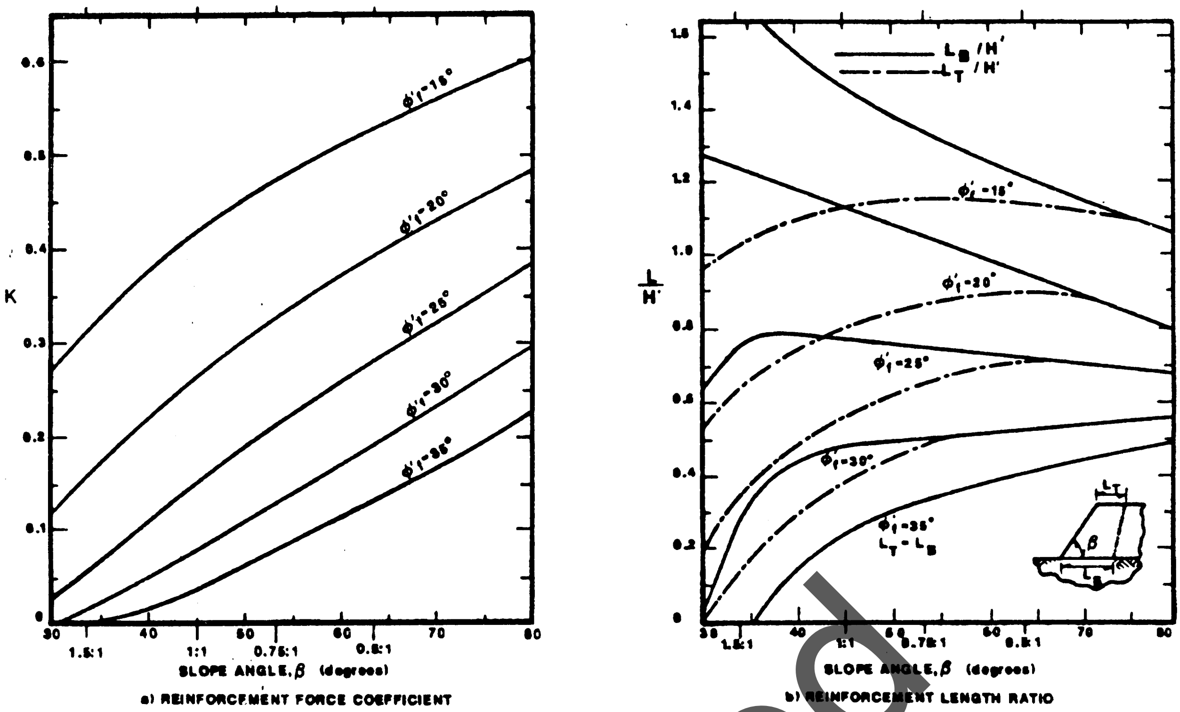


CHART PROCEDURE:

1) Determine force coefficient K from figure above, where ϕ_r = friction angle of reinforced fill:

$$\phi_f = \tan^{-1}\left(\frac{\tan\phi_r}{FS_R}\right)$$

2) Determine:

$$T_{S\text{-}MAX} = 0.5\ K\ \gamma_r\ (H')^2$$

where: $H' = H + q/\gamma_r$
q = a uniform load

3) Determine the required reinforcement length at the top L_T and bottom L_B of the slope from the figure above.

LIMITING ASSUMPTIONS

- Extensible reinforcement.
- Slopes constructed with uniform, cohesionless soil, c = 0).
- No pore pressures within slope.
- Competent, level foundation soils.
- No seismic forces.
- Uniform surcharge nor greater than 0.2 γ_r H.
- Relatively high soil/reinforcement interface friction angle, $\phi_{sg} = 0.9\ \phi_r$ (may not be appropriate for some geotextiles).

Figure 8-6 Sliding wedge approach to determine the coefficient of earth pressure K (after Schmertmann, et al., 1987). NOTE: Charts © The Tensar Corporation.

d. Determine reinforcement vertical spacing S_v or the maximum design tension T_{max} requirements.

For each zone, calculate the design tension, T_{max}, requirements for each reinforcing layer based on an assumed S_v. If the allowable reinforcement strength is known, calculate the minimum vertical spacing and number of reinforcing layers, N, required for each zone based on:

$$T_{max} = \frac{T_{zone}\, S_v}{H_{zone}} = \frac{T_{zone}}{N} \leq T_{al}\, R_c$$

where:

R_c = percent coverage of reinforcement, in plan view (R_c = 1 for continuous sheets);

S_v = vertical spacing of reinforcement; should be multiples of compaction layer thickness for ease of construction;

T_{zone} = maximum reinforcement tension required for each zone; T_{zone} equals $T_{S\text{-}MAX}$ for low slopes (H < 6 m);

H_{zone} = height of zone, and is equal to T_{top}, T_{middle}, and T_{Bottom} for high slopes (H > 6 m); and

N = number of reinforcement layers.

- Short (1.2 to 2 m) lengths of intermediate reinforcement layers can be used to maintain a maximum vertical spacing of 600 mm or less for face stability and compaction quality (Figure 8-7).

  - For slopes less than 1H:1V, closer spaced reinforcements (*i.e.*, every lift or every other lift, but no greater than 400 mm) generally preclude having to wrap the face. Wrapped faces are usually required for steeper slopes to prevent face sloughing. Alternative vertical spacings can be used to prevent face sloughing, but in these cases a face stability analysis should be performed using the method presented in Section 8.3 or as recommended by Thielen and Collin (1993).
  - Intermediate reinforcement should be placed in continuous layers and need not be as strong as the primary reinforcement, but in any case, all reinforcements should be strong enough to survive installation (*e.g.*, see Tables 5-1 and 5-2).

e. To ensure that the rule-of-thumb reinforcement force distribution is adequate for critical or complex structures, recalculate T_S using the equation in Step 6a for potential failure directly above each layer of primary reinforcement.

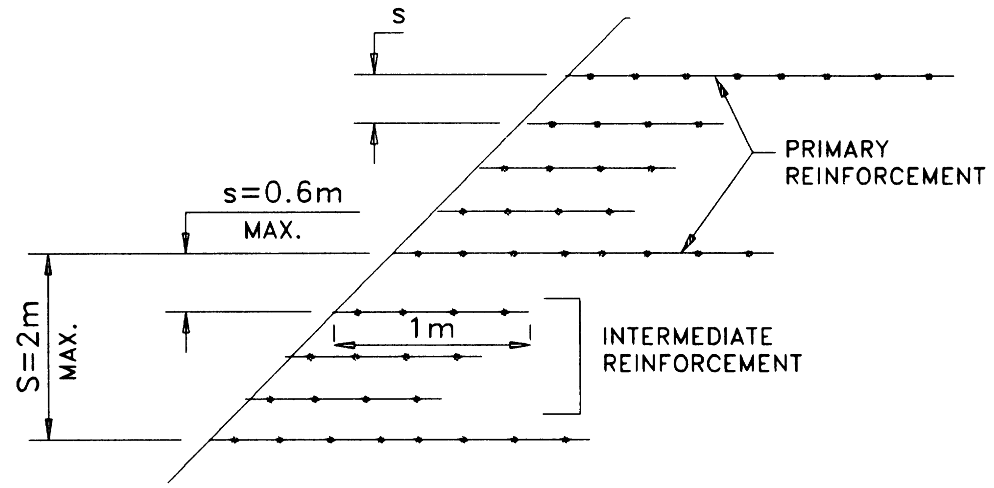


Figure 8-7 Spacing and embedding requirements for slope reinforcement showing: (a) primary and intermediate reinforcement layout; and (b) uniform reinforcement layout.

f. Determine the reinforcement lengths required.

- The embedment length, L_e, of each reinforcement layer beyond the most critical sliding surface found in Step 6a (*i.e.*, circle found for $T_{S\text{-}MAX}$) must be sufficient to provide adequate pullout resistance. For the method illustrated in Figure 8-5, use:

$$L_e = \frac{T_{max} \; FS}{2 \; F^* \; \alpha \; \sigma'_v}$$

where:

F^*, α , and σ'_v are defined in Section 8.4 and Appendix K.

- Minimum value of L_e is 1 m. For cohesive soils, check L_e for both short- and long-term pullout conditions. For long-term design, use ϕ'_r with $c_r = 0$. For short-term evaluation, conservatively use ϕ_r and $c_r = 0$ from consolidated-undrained tests, or run pullout tests.
- Plot the reinforcement lengths as obtained from the pullout evaluation on a cross section containing the rough limits of the critical zone determined in Step 5.
  - The length required for sliding stability at the base will generally control the length of the lower reinforcement levels.

- Lower layer lengths must extend to the limits of the critical zone. Longer reinforcements may be required to resolve deep seated failure problems.
- Upper levels of reinforcement may not have to extend to the limits of the critical zone provided sufficient reinforcement exists in the lower levels to provide FS_R for all circles within the critical zone (*e.g.*, see Step 6g).

- Check that the sum of the reinforcement forces passing though each failure surface is greater than T_s, from Step 6a, required for that surface.
  - Only count reinforcement that extends 1 meter beyond the surface to account for pullout resistance.
  - If the available reinforcement resistance is not sufficient, increase the length of reinforcement not passing through the surface or increase the strength of lower-level reinforcement.

- Simplify the layout by lengthening some reinforcement layers to create two or three sections of equal reinforcement length for ease of construction and inspection.

- Reinforcement layers generally do not need to extend to the limits of the critical zone, except for the lowest levels of each reinforcement section.

- Check the obtained length using Chart B in Figure 8-6. Note: L_e is already included in the total length, L_t and L_B from Chart B.

g. Check design lengths of complex designs.

- When checking a design that has differing reinforcement length zones, lower zones may be over-reinforced to provide reduced lengths of upper reinforcement levels.

- In evaluating the length requirements for such cases, the reinforcement pullout stability must be carefully checked in each zone for the critical surfaces exiting at the base of each length zone.

STEP 6. Method B - Trial reinforcement analysis.

Another way to design reinforcement for a stable slope is to develop a trial layout of reinforcement and analyze the reinforced slope with a computer program, such as the new FHWA program RSS. Layout includes number, length, design strength, and vertical distribution of the geosynthetic reinforcement. The charts presented in Figure

8-6 provide a method for generating a preliminary layout. Note that these charts were developed with the specific assumptions noted on the figure.

Analyze the RSS with trial geosynthetic reinforcement layouts. The most economical reinforcement layout will be one which results in approximately equal, but greater than the minimum required, stability safety factor for internal, external, and compound failure planes. A contour plot of lowest safety factor values about the trial failure circle centroids is recommended to *map* and locate the minimum safety factors for the three modes of failure.

External stability analysis in Step 7 will then include an evaluation of local bearing capacity, foundation settlement, and dynamic stability.

STEP 7. Check external stability.

The external stability of a reinforced soil mass depends on the soil mass's ability to act as a stable block and withstand all external loads without failure. Failure possibilities include sliding, deep-seated overall instability, local bearing capacity failure at the toe (lateral squeeze-type failure), as well as compound failures initiating internally and externally through the short- and long-term conditions.

a. Sliding resistance.

The reinforced mass must be wide enough at any level to resist sliding along the reinforcement. A wedge-type failure surface defined by the reinforcement limits (the length of the reinforcement from the toe) identified in Step 5 can be checked to ensure it is sufficient to resist sliding from the following relationships:

$$\text{Resisting Force} = \text{FS x Sliding Force}$$
$$(W + P_a \sin \phi_b) \tan \phi_{min} = FS\, P_a \cos \phi_b$$

with:

$W = \frac{1}{2} L^2 \gamma_r (\tan \beta_r)$ for $L < H$

$W = [LH - H^2/(2\tan\beta)] (\gamma_r)$ for $L > H$

$P_a = \frac{1}{2} \gamma_b H^2 K_a$

where:

L = length of bottom reinforcing layer in each zone where there is a reinforcement length change;

H = height of slope;

FS	=	factor of safety for sliding (> 1.5);
P_A	=	active earth pressure;
ϕ_{min}	=	minimum angle of shearing friction either between reinforced soil and geosynthetic or the friction angle of the foundation soil;
β	=	slope angle;
γ_r & γ_b	=	unit weight of the reinforced and retained backfill, respectively; and
ϕ_b	=	friction angle of retained fill (Note: If geotextile filter or geocomposite drains are placed continuously on the backslope, then ϕ_b should be set equal to the interface friction angle between the geosynthetic and the retained fill).

b. Deep-seated global stability.

As a check, potential deep-seated failure surfaces behind the reinforced soil mass should be reevaluated. The analysis performed in Step 5 should provide this information. However, as a check, classical rotational slope stability methods such as simplified Bishop (1955), Morgenstern and Price (1965), Spencer (1981), or others, may be used. Appropriate computer programs also may be used.

c. Local bearing failure at the toe (lateral squeeze).

Consideration must be given to the bearing capacity at the toe of the slope. High lateral stresses in a confined soft stratum beneath the embankment could lead to a lateral squeeze-type failure. This must be analyzed if the slope is on a soft — not firm foundation. Refer to Chapter 7, or Elias and Christopher (1997) for design equations and references.

d. Foundation settlement.

The magnitude of foundation settlement should be determined using ordinary geotechnical engineering procedures (see FHWA *Soils and Foundations Workshop Manual* (Cheney and Chassie, 1993)). If the calculated settlement exceeds project requirements, then foundation soils must be improved.

e. Dynamic stability.

- Perform a pseudo-static type analysis using a seismic ground coefficient A_o, obtained from local building code and a design seismic acceleration A_m equal to $A_o/2$ (Elias and Christopher, 1997).

- If the slope is located in an area subject to potential seismic activity, then some type of dynamic analysis is warranted. Usually a simple pseudo-static type analysis is carried out using a seismic coefficient obtained from local or national codes. For critical projects in areas of potentially high seismic risk, a complete dynamic analysis should be performed (Berg, 1993).

- The liquefaction potential of the foundation soil should also be evaluated.

STEP 8. Evaluate requirements for subsurface and surface water control.

a. Subsurface water control.

Uncontrolled subsurface water seepage can decrease slope stability and ultimately result in slope failure. Hydrostatic forces on the rear of the reinforced mass and uncontrolled seepage into the reinforced mass will decrease stability. Seepage through the mass can reduce pullout capacity of the geosynthetic and create erosion at the face. Consider the water source and the permeability of the natural and fill soils through which water must flow when designing subsurface water drainage features.

- Design of subsurface water drainage features should address flow rate, filtration, placement, and outlet details.

- Drains are typically placed at the rear of the reinforced mass. Lateral spacing of outlets is dictated by site geometry, expected flow, and existing agency standards. Geocomposite drainage systems or conventional granular blanket and trench drains could be used.

- Lateral spacing of outlet is dictated by site geometry, estimated flow, and existing agency standards. Outlet design should address long-term performance and maintenance requirements.

- The design of geocomposite drainage materials is addressed in Chapter 2.

- Slope stability analyses should account for interface shear strength along a geocomposite drain. The geocomposite/soil interface will most likely have a friction value that is lower than that of the soil. Thus, a potential failure surface may be induced along the interface.

- Geotextiles reinforcements (primary and intermediate layers) must be more permeable than the reinforced fill material to prevent a hydraulic build-up above the geotextile layers during precipitation.

Special emphasis on the design and construction of subsurface drainage features is recommended for structures where drainage is critical for maintaining slope stability. Redundancy in the drainage system is also recommended in these cases.

b. Surface water runoff.

Slope stability can be threatened by erosion due to surface water runoff. Erosion rills and gullies can lead to surface sloughing and possibly deep-seated failure surfaces. Erosion control and revegetation measures must, therefore, be an integral part of all reinforced slope system designs and specifications.

Surface water runoff should be collected above the reinforced slope and channeled or piped below the base of the slope. Standard agency drainage details should be utilized.

If not otherwise protected, reinforced slopes should be vegetated after construction to prevent or minimize erosion due to rainfall and runoff on the face. Vegetation requirements will vary by geographic and climatic conditions and are therefore project-specific. Geosynthetic reinforced slopes are inherently difficult sites to establish and maintain vegetative cover due to these steep slopes. The steepness of the slope limits the amount of water absorbed by the soil before runoff occurs. Once vegetation is established on the face, it must be maintained to ensure long-term survival.

A synthetic (permanent) erosion control mat that is stabilized against ultraviolet light and is inert to naturally occurring soil-born chemicals and bacteria may be required with seeding. The erosion control mat serves three functions: 1) to protect the bare soil face against erosion until vegetation is established, 2) to reduce runoff velocity for increased water absorption by the soil, thereby promoting long-term survival of the vegetative cover, and 3) to reinforce the root system of the vegetative cover. Maintenance of vegetation will still be required.

A permanent synthetic mat may not be required in applications characterized by flatter slopes (less than 1:1), low height slopes, and/or moderate runoff. In these cases, a temporary (degradable) erosion blanket may be specified to protect the slope face and promote growth until vegetative cover is firmly established. Refer to Chapter 4 for design of erosion mats and blankets.

Erosion control mats and blankets vary widely in type, cost, and — more importantly — applicability to project conditions. **Slope protection should not be left to the Contractor's or vendor's discretion.** Guideline material specifications are provided in Section 8.8. For additional design guidance, see Elias and Christopher (1997).

8.4 MATERIAL PROPERTIES

8.4-1 Reinforced Slope Systems

Reinforced soil systems consist of planar reinforcements arranged in horizontal planes in the fill soil to resist outward movement of the reinforced soil mass. Facing treatments ranging from vegetation to flexible armor systems are applied to prevent raveling and sloughing of the face. These systems are generic in nature and can incorporate any of a variety of reinforcements and facing systems. This section provides the material properties required for design.

8.4-2 Soils

Any soil meeting the requirements for embankment construction can be used in a reinforced slope system. From a reinforcement point of view alone, even lower-quality soil than that conventionally used in unreinforced slope construction could be used; however, a higher-quality material offers less durability concerns and is easier to handle, place, and compact, which tends to speed up construction. Therefore, the following guidelines are provided as recommended backfill requirements for reinforced engineered slopes.

Gradation (Christopher et al., 1990): Recommended backfill requirements for reinforced engineered slopes are:

Sieve Size	Percent Passing
20 mm*	100 - 75
4.75 mm	100 - 20
0.425 mm	0 - 60
0.075 mm	0 - 50

Plasticity Index (PI) $\leq$ 20 (AASHTO T-90)

Soundness: Magnesium sulfate soundness loss less than 30% after 4 cycles, based on AASHTO T 104 or equivalent sodium sulfate soundness of less than 15 percent after 5 cycles.

* The maximum size can be increased up to 100 mm provided field tests have been, or will be, performed to evaluate potential strength reduction due to

installation damage (see Appendix K). In any case, geosynthetic strength reduction factors for site damage should be checked in relation to particle size and angularity of the larger particles.

Definition of total and effective stress shear strength properties becomes more important as the percentage passing the 0.075 mm sieve increases. Likewise, drainage and filtration design are more critical. Fill materials outside of these gradation and plasticity index requirements have been used successfully (Christopher et al., 1990; Hayden et al.,1991); however, long-term (> 5 years) performance field data is not available. Performance monitoring is recommended if fill soils fall outside of the requirements listed above.

Chemical Composition (Elias and Christopher, 1997): The chemical composition of the fill and retained soils should be assessed for effect on reinforcement durability (primarily pH and oxidation agents). Some of the soil environments posing potential concern when using geosynthetics are listed in Appendix K. Tentatively, use polyester geosynthetics should be limited to soils with $3 < pH > 9$; and polyolefins (polypropylene and polyethylene) should be limited to soils with $pH > 3$. Soil pH should be determined in accordance with AASHTO T-289.

Compaction (Christopher et al., 1990): Soil fill shall be compacted to 95% of optimum dry density (γ_d) and + or - 2 % of the optimum moisture content, w_{opt}, according to AASHTO T-99. Cohesive soils should be compacted in 150 to 200 mm compacted lifts, and granular soils in 200 to 300 mm compacted lifts.

Shear Strength (Berg, 1993): Peak shear strength parameters determined using direct shear or consolidated-drained (CD) triaxial tests should be used in the analysis (Christopher et al., 1990). Effective stress strength parameters should be used for granular soils with less than 15% passing the 0.075 mm sieve.

For all other soils, peak effective stress and total stress strength parameters should be determined. These parameters should be used in the analyses to check stability for the immediately-after-construction and long-term cases. Use CD direct shear tests (sheared slowly enough for adequate sample drainage), or consolidated-undrained (CU) triaxial tests with pore water pressures measured for determination of effective stress parameters. Use CU direct shear or triaxial tests for determination of total stress parameters.

Shear strength testing is recommended. However, use of assumed shear values based on Agency guidelines and experience may be acceptable for some projects. Verification of site soil type(s) should be completed following excavation or identification of borrow pit, as applicable.

Unit Weights: Dry unit weight for compaction control, moist unit weight for analyses, and saturated unit weight for analyses (where applicable) should be determined for the fill soil.

8.4-3 Geosynthetic Reinforcement

Geosynthetic design strength must be determined by testing and analysis methods that account for long-term interaction (*e.g.*, grid/soil stress transfer) and durability of the all geosynthetic components. Geogrids transfer stress to the soil through passive soil resistance on the grid's transverse members and through friction between the soil and the geogrid's horizontal surfaces (Mitchell and Villet, 1987). Geotextiles transfer stress to the soil through friction.

An inherent advantage of geosynthetics is their longevity in fairly aggressive soil conditions. The anticipated half-life of some geosynthetics in normal soil environments is in excess of 1000 years. However, as with steel reinforcements, strength characteristics must be adjusted to account for potential degradation in the specific environmental conditions, even in relatively neutral soils. Questionable soil environments are listed in Appendix K.

Tensile Strengths: Long-term tensile strength (T_{al}) of the geosynthetic shall be determined using a partial factor of safety approach (Bonaparte and Berg, 1987). Reduction factors are used to account for installation damage, chemical and biological conditions and to control potential creep deformation of the polymer. Where applicable, a reduction is also applied for seams and connections. The total reduction factor is based upon the mathematical product of these factors. The long-term tensile strength, T_{al}, thus can be obtained from:

$$T_{al} = \frac{T_{ult}}{RF}$$

with RF equal to the product of all applicable reduction factors:

$$RF = RF_{CR} \times RF_{ID} \times RF_{D}$$

where:

T_{al} = long-term geosynthetic tensile strength,(kN/m);
T_{ult} = ultimate geosynthetic tensile strength, based upon MARV, (kN/m);
RF_{CR} = creep reduction factor, ratio of T_{ult} to creep-limiting strength, (dimensionless);
RF_{ID} = installation damage reduction factor, (dimensionless); and
RF_{D} = durability reduction factor for chemical and biological degradation, (dimensionless).

RF values for durable geosynthetics in non-aggressive, granular soil environments range from 5 to 7. Appendix K suggests that a default value RF = 7 may be used for routine, non-critical structures **which meet the soil, geosynthetic and structural limitations listed in the appendix.** However, as indicated by the range of RF values, there is a potential to significantly reducing the reinforcing requirements and the corresponding cost of the structure by obtaining a reduced RF from test data.

The procedure presented above and detailed in Appendix K is derived from Elias and Christopher (1997), Berg (1993), the Task Force 27 (1990) guidelines for geosynthetic reinforced soil retaining walls, the Geosynthetic Research Institute's Methods GG4a and GG4b - *Standard Practice for Determination of the Long Term Design Strength of Geogrids* (1990, 1991), and the Geosynthetic Research Institute's Method GT7 - *Standard Practice for Determination of the Long Term Design Strength of Geotextiles* (1992).

For RSS structures, the FS value will be dependent upon the analysis tools utilized by the designer. With computerized analyses, the FS value is dependent upon how the specific program accounts for the reinforcement tension is computing a stability factor of safety.

The method of analysis in section 8.3 and in FHWA's *RSS* program, as well as many others, assume the reinforcement force as contributing to the resisting moment, *i.e.*:

$$FS_R = \frac{M_R + T_S\,R}{M_D}$$

With this assumption, the allowable strength is equal to the long-term strength (if no reduction is required for seams or joints). The factor of safety on the reinforcement is equal to the stability factor of safety required (FS_R) and $T_a = T_{al}$ as was used in section 8.3-2.

Some computer programs use an assumption that the reinforcement force is a negative driving component, thus the FS is computed as:

$$FS = \frac{M_R}{M_D - T_S\,R}$$

With this assumption, the stability factor of safety is not applied to T_S. Therefore, the allowable strength should be computed as the long-term strength divided by the required safety factor (i.e., target stability factor of safety). That is:

$$T_a = \frac{T_{al}}{FS_R}$$

This provides an appropriate factor of safety for uncertainty in material strengths and reduction factors. The method used to develop design charts should likewise be carefully evaluated to determine FS used to obtain the allowable geosynthetic strength.

Soil-Reinforcement Interaction: Two types of soil-reinforcement interaction coefficients or interface shear strengths must be determined for design: pullout coefficient, and interface friction coefficient (Task Force 27 Report, 1990). Pullout coefficients are used in stability analyses to compute mobilized tensile force **at the front** and **tail** of each reinforcement layer. Interface friction coefficients are used to check factors of safety against outward sliding of the entire reinforced mass.

Detailed procedures for quantifying interface friction and pullout interaction properties are presented in Appendix K. The ultimate pullout resistance, P_r, of the reinforcement per unit width of reinforcement is given by:

$$P_r = 2 \bullet F^* \bullet \alpha \bullet \sigma'_v \bullet L_e$$

where:

$L_e \bullet 2$	=	the total surface area per unit width of the reinforcement in the resistance zone behind the failure surface
L_e	=	the embedment or adherence length in the resisting zone behind the failure surface
F^*	=	the pullout resistance (or friction-bearing-interaction) factor
α	=	a scale effect correction factor
σ'_v	=	the effective vertical stress at the soil-reinforcement interfaces

For preliminary design in the absence of specific geosynthetic test data, and for standard backfill materials with the exception of uniform sands (i.e., coefficient of uniformity, $C_u < 4$), it is acceptable to use conservative default values for F* and α as shown in Table 8-1.

TABLE 8-1 DEFAULT VALUES FOR F* AND α PULLOUT FACTORS

Reinforcement Type	Default F*	Default α
Geogrid	0.8 tan ϕ	0.8
Geotextile	0.67 tan ϕ	0.6

8.5 PRELIMINARY DESIGN AND COST EXAMPLE

EVALUATION AND COST ESTIMATE EXAMPLE

A 1 kilometer long, 5 m high, 2.5H:1V side slope road embankment in a suburban area is to be widened by one lane. At least a 6 m width extension is required to allow for the additional lane plus shoulder improvements. Several options are being considered.

1. Simply extend the slope of the embankment.

2. Construct a 2.5 m high concrete, cantilever retaining wall at the toe of the slope, extend the alignment, and slope down at 2.5H:1V to the wall. (Of course a geosynthetically reinforced retaining wall should also be considered, but that's covered in the next chapter.)

3. Construct a 1H:1V reinforced soil slope up from the toe of the existing slope, which will add 7.5 m to the alignment, enough for future widening, if required.

Guardrails are required for all options and is not included in the cost comparisons.

Option 1

The first alternative will require 30 m^3 fill per meter of embankment length. The fill is locally available with some hauling required and has an estimated in-place cost of $\$8/m^3$ (about \$4.00 per 1000 kg). The cost of the 6 m right of way is $\$15/m^2$, for a cost of \$90 per meter of embankment length. Finally, hydroseeding and mulching will cost approximately \$0.75 per meter of face, or approximately \$10 per meter of embankment. Thus the total cost of embankment will be \$340 for the full height per meter length of embankment or $\$68/m^2$ of vertical face. There will also be a project delay while the additional right of way is obtained.

Option 2

Based on previous projects in this area, the concrete retaining wall is estimated to cost $\$400/m^2$ of vertical wall face including backfill. Thus, the 2.5 m high wall will cost \$1,000 per meter length of embankment. This leads to a cost of $\$200/m^2$ of vertical embankment per meter length of structure. In addition 18 m^3 of fill will be required to construct the sloped portion, adding \$144 per meter of embankment, or $\$28.80/m^2$ of vertical face, to the cost. Hydroseeding and mulching of the slope will add about a $\$1/m^2$ of vertical face to the cost. Thus, the total cost of this option is estimated at $\$230/m^2$ of vertical embankment face. This option will require an additional 2 weeks of construction time to allow the concrete to cure. On some projects, additional costs can be incurred due to the delay plus additional traffic control and highway personnel required for inspection during removal of the forms. Since this project was part of a larger project, such delays were not considered.

Option 3

This option will require a preliminary design to determine the quantity of reinforcement.

STEP 1. Slope description

a. $H = 5$ m
b. $\beta = 45°$
c. $q = 10$ kPa (for pavement section) + 2% road grade

Performance requirements

a. External Stability:
Sliding Stability: $FS_{min} = 1.3$
Overall slope stability and deep seated: $FS_{min} = 1.3$

Dynamic loading: no requirement
Settlement: analysis required

b. Compound Failure: $FS_{min} = 1.3$
c. Internal Stability: $FS_{min} = 1.3$

STEP 2. Engineering properties of foundation soils.

a. Review of soil borings from the original embankment construction indicates foundation soils consisting of stiff to very stiff, low-plasticity silty clay with interbedded seams of sand and gravel. The soils tend to increase in density and strength with depth.
b. $\gamma_d = 19$ kN/m^3, $\omega_{opt} = 15\%$ UU = 100 kPa, $\phi' = 28°$, $c' = 0$
c. At the time of the borings, $d_w = 2$ m below the original ground surface.
d. Not applicable

STEP 3. Properties of reinforced and embankment fill
(The existing embankment fill is a clayey sand and gravel). For preliminary evaluation, the properties of the embankment fill are assumed for the reinforced section as follows:

a.

Sieve Size	Percent Passing
100 mm	100%
20 mm	99%
4.75 mm	63%
0.425 mm	45%
0.075 mm	25%

PI (of fines) = 10
Gravel is competent
pH = 7.5

b. $\gamma_r = 21$ kN/m^3, $\omega_{opt} = 15\%$
c. $\phi' = 33°$, $c' = 0$
d. Soil is relatively inert

STEP 4. Design parameters for reinforcement
For preliminary analysis use default values.

a. $T_{al} = T_{ult}/R_f$
b. $FS_{po} = 1.5$

STEP 5. Check unreinforced stability

Using STABL5M, the minimum unreinforced factor of safety was 0.68 with the critical zone defined by the target factor of safety FS_R as shown in Design Figure A.

STEP 6. Calculate T_S for the FS_R

Option A. From the computer runs, obtain FS_U, M_D and R for each failure surface within the critical zone, and calculate T_S as follows. (NOTE: With minor code modification, this could easily be done as part of the computer analysis, as is done in the FHWA program RSS.)

a.

$$T_S = (1.3 - FS_U)\frac{M_D}{R}$$

Evaluating all of the surfaces in the critical zone indicates maximum $T_{S\text{-}MAX}$ = 49.7 kN/m for FS_U = 0.89 as shown in Design Figure B.

b. Checking $T_{S\text{-}MAX}$ by using Figure 8-6:

$$\phi_f = \tan^{-1}\left(\frac{\tan\phi_r}{FS_R}\right) = \tan^{-1}\left(\frac{\tan 33^o}{1.3}\right) = 26.5^o$$

From Figure 8-6, K ≈ 0.14

and,

$$H' = H + q/\gamma_r + 0.1 \text{ m (for 2\% road grade)}$$
$$= 5 \text{ m} + (10 \text{ kN/m}^2 \div 21 \text{ kN/m}^3) + 0.1 \text{ m} = 5.6 \text{ m}$$

then,

$$T_{S\text{-}MAX} = 0.5\, K\, \gamma_r\, H'$$
$$= 0.5\,(0.14)\,(21\, kN/m^3)\,(5.6\, m)^2$$
$$= 46.1\ kN/m$$

The evaluation using Figure 8-6 appears to be in good agreement with the computer analysis.

c. Determine the distribution of reinforcement.

Since H < 6m, use a uniform spacing. Due to the cohesive nature of the backfill, maximum compaction lifts of 200 mm are recommended.

d. To avoid wrapping the face, use S_v = 400 mm reinforcement spacing; therefore, N = 5m/0.4m = 12.5. Use 12 layers with the bottom layer placed after the first lift of embankment fill.

$$T_d = \frac{T_{\max}}{N} = \frac{49.7\ kN/m}{12} = 4.14\ kN/m$$

(NOTE: Other reinforcement options such as using short secondary reinforcements at every lift with spacing and strength increased for primary reinforcements could be considered, and should be evaluated, for selecting the most cost-effective option for final design.)

e. Since this is a simple structure, rechecking T_s above each layer of reinforcement is not performed.

f. For preliminary analysis, the critical zone found in the computer analysis (Figure A) can be used to define the reinforcement limits. This is especially true for this problem, since the factor of safety for sliding ($FS_{sliding}$ ≥ 1.3) is greater than the internal stability requirement ($FS_{internal}$ ≥ 1.3); thus, the sliding failure surface well encompasses the most critical reinforcement surface.

As measured at the bottom and top of the sliding surface in Figure A, the required lengths of reinforcement are: L_{bottom} = 5.3 m
L_{top} = 2.9 m

Check length of embedment beyond the critical surfce L_e and factor of safety against pullout.

Since the most critical location for pullout is the reinforcement near the top of the slope (depth Z = 0.2 m), subtract the distance from the critical surface to the face of the slope in Figure B from L_{top}. This gives L_e at top = 1.3 m.

Assuming the most conservative assumption for pullout factors F* and α from section 8.4 and Appendix K gives F* = 0.67 tanϕ and α = 0.6. Therefore,

$$FS_{PO} = \frac{L_e\, F^*\, \alpha\, \sigma_v\, C}{T_{max}} = \frac{1.3\ (\ 0.67\ tan33^\circ\ )\ (\ 0.6\ )\ (\ 0.2\ m\ x\ 21\ kN/m^3\ +\ 10\ kN/m^2)\ (\ 2)}{4.14\ kN/m}$$

$FS_{PO} = 2.3 > 1.5$ required

Check the length requirement using Figure 8-6. For L_B

$$\phi_f = \tan^{-1}(\frac{\tan 28^o}{1.3}) = 22.2^o$$

From Figure 8-6: $L_B/H' = 0.96$
thus, $L_B = 5.6$ m (0.96) = 5.4 m

For L_T

$$\phi_f = \tan^{-1}(\frac{\tan 33^o}{1.3}) = 26.5^o$$

From Figure 8-6: $L_T/H' = 0.52$
thus, $L_T = 5.6$ m (0.52) = 2.9 m

Using Figure 8-6, the evaluation again appears to be in good agreement with the computer analysis.

g. This is a simple structure and additional evaluation of design lengths is not required.

Option B. Since this is a preliminary analysis and a fairly simple problem, Figure 8-6 or any number of proprietary computer programs, can be used to rapidly evaluate T_s and T_d.

In summary, 12 layers of reinforcement are required with a design strength, T_d, of 4.14 kN/m and an average length of 5 m over the full height of embankment. This would result in a total of 60 m^2 reinforcement per meter length of embankment or 12 m^2 per vertical meter of height. Adding 10% to 15% for overlaps and overages results in an anticipated reinforcement volume of 13.5 m^2 per vertical embankment face. Based on the cost information in Appendix K, reinforcement with an allowable strength $T_a \geq 4.14$ kN/m would cost approximately \$1.00 to \$1.50/m^2. Assuming \$0.50 m^2 for handling and placement, the in-place cost of reinforcement would be approximately \$25/m^2 of vertical embankment face. Approximately 18.8 m^3 of additional backfill would be required for this option, adding \$30/m^2 to the cost of this option. In addition, overexcavation and backfill of existing embankment material will be required to allow for reinforcement placement. Assuming \$2/m^3 for overexcavation and replacement will add approximately \$4/m^2 of vertical face. Erosion protection for the face will also add a cost of \$5/m^2 of vertical face. Thus, the total estimated cost for this option totals approximately \$64/m^2 of vertical embankment face.

Option 3 provides a slightly lower cost than Option 1 plus it does not require additional right-of-way.

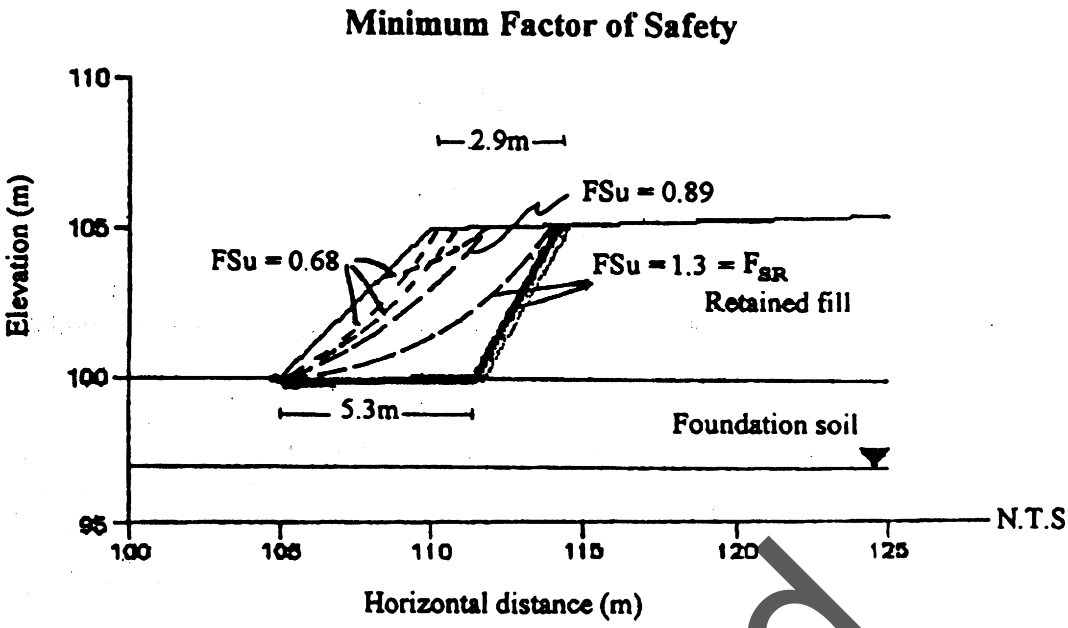


Figure A Unreinforced stability analysis.

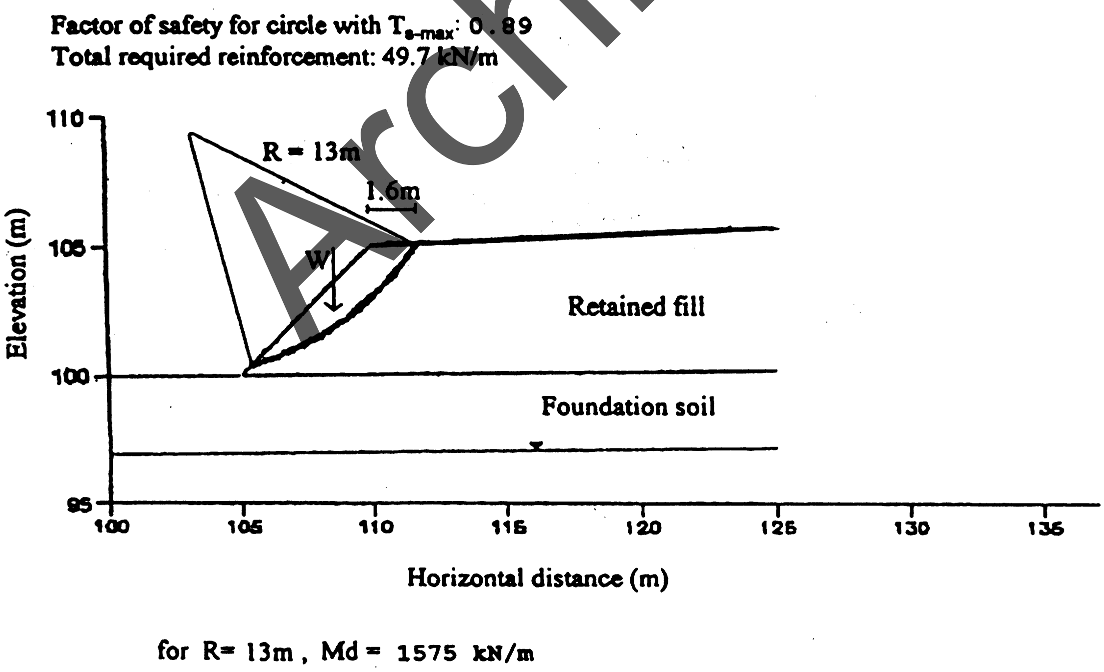


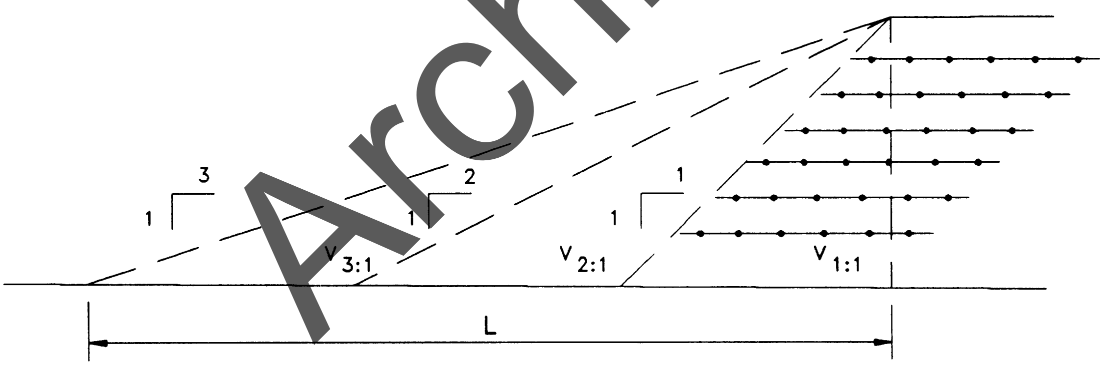
Figure B Surface requiring maximum reinforcement (*i.e.*, most critical reinforced surface).

8.6 COST CONSIDERATIONS

As with any other reinforcement application, an appropriate benefit-to-cost ratio analysis should be completed to determine if the steeper slope with reinforcement is justified economically over the flatter slope with increased right-of-way and materials costs, etc. In some cases, however, the height of the embankment will be controlled by grade requirements, and the slope might as well be as steep as possible. With respect to economy, the factors to consider are:

- cut or fill earthwork quantities;
- size of slope area;
- average height of slope area;
- angle of slope;
- cost of nonselect versus select backfills;
- erosion protection requirements;
- cost and availability of right-of-way needed;
- complicated horizontal and vertical alignment changes;
- safety equipment (guard rails, fences, etc.)
- need for temporary excavation support systems;
- maintenance of traffic during construction; and
- aesthetics.

Figure 8-8 provides a rapid first order assessment of cost for comparing a flatter unreinforced slope with a steeper reinforced slope.



$V_{3:1} = V$	$V_{3:1} = L$
$V_{2:1} = \frac{2}{3}V$	$V_{2:1} = \frac{2}{3}L$
$V_{1:1} = \frac{1}{3}V$	$V_{1:1} = \frac{1}{3}L$

<u>COST:</u>

3H:1V = V_{SOIL} + L_{LAND} + Guardrail (?) + Hydroseeding(?)

2H:1V = $\frac{2}{3}V_{SOIL}$ + $\frac{2}{3}L_{LAND}$ + Guardrail + Erosion Control + High Maintenance

1H:1V = $\frac{1}{3}V_{SOIL}$ + $\frac{1}{3}L_{LAND}$ + Reinforcement + Guardrail + Erosion Control

Figure 8-8 Cost evaluation of reinforced soil slopes.

8.7 IMPLEMENTATION

The recent availability of many new geosynthetic reinforcement materials -- as well as drainage and erosion control products -- requires Engineers to consider many alternatives before preparing contract bid documents so that proven, cost-effective materials can be chosen. Reinforced soil slopes may be contracted using two different approaches. Slope structures can be contracted on the basis of (Berg, 1993):

- In-house (Agency) design with geosynthetic reinforcement, drainage details, erosion measures, and construction execution specified.
- System or end-result approach using approved systems, similar to mechanically stabilized earth (MSE) walls, with lines and grades noted on the drawings.

For either approach, the following assumptions should be considered:

- Geosynthetic reinforced slope systems can successfully compete with select embankment fill requirements, other landslide stabilization techniques, and unreinforced embankment slopes in urban areas.
- Value engineering proposals are allowed, based on Agency standard procedures. Geosynthetic reinforced slope systems submitted for use in a value engineering proposal should have previous approval.
- Though they may incorporate proprietary materials, reinforced slope systems are non-proprietary and may be bid competitively with geosynthetic reinforcement material alternatives. Geosynthetic reinforcement design parameters must be based upon documentation that is provided by the manufacturer, submitted and approved by the Agency, or based upon default partial safety values as described in Section 8.3 and Appendix K.
- Designers contemplating the use of reinforced slope systems should offer the same degree of involvement to all suppliers who can accomplish the project objectives.
- Geosynthetic reinforcement material specifications and special provisions for reinforced slope systems should require suppliers to provide a qualified and experienced representative at the site, for a minimum of three days, to assist the contractor and Agency inspectors at the start of construction. If there is more than one slope on a project, then this criteria should apply to construction of the initial slope only. From then on, the representative should be available on an as-needed basis, **as requested by the Agency Engineer**, during construction of the remainder of the slope(s).
- An in-house design approach and an end result approach to reinforced slope solicitation are included in this document. Some user agencies prefer one approach over the other, or a mixed use of approaches depending on the criticalness of the slope structures. **Both approaches are acceptable** if properly implemented. Each approach has advantages and disadvantages.

Any proprietary material should undergo an Agency review prior to inclusion as an alternate offered either during the design (in-house) or construction (value engineering or end result) phase.

It is highly recommended that each Transportation Agency develop documented procedures for the following.

- Review and approval of geosynthetic soil reinforcing materials.
- Review and approval of drainage composite materials.
- Review and approval of erosion control materials.
- Review and approval of geosynthetic reinforced slope systems and suppliers (geosynthetic soil reinforcing materials, drainage composites, and erosion control materials).
- In-house design of reinforced slopes.
- Contracting for outside design and supply of reinforced slope systems.

Guidelines for preparation of geosynthetic review and approval documents are included in *Guidelines for Design, Specification, and Contracting of Geosynthetic Mechanically Stabilized Earth Slopes on Firm Foundations* (Berg, 1993).

8.8 SPECIFICATIONS AND CONTRACTING APPROACH

This section includes:

- requirements for the specification of reinforcement materials based on **in-house design**; and
- a guideline for the (**performance**) specification of a reinforced slope system (line and grade), based on conceptual plans developed by the agency.

Conceptual plans must contain the geometric, geotechnical, and project-specific design information. A framework is provided for developing an Agency specification or special provision. Generic pullout and allowable tensile strength values, based upon preapproved geotextiles or geogrids should be inserted into the corresponding Tables 8-2 and 8-3.

Reinforced slopes may be designed in-house or by a geosynthetic reinforcement supplier based upon a performance specification. Geosynthetic reinforcement, erosion measures, and drainage details must be specified when contracting based on an in-house design. Permanent synthetic mats may be required on steeper and taller slopes, and degradable blankets may be sufficient for flatter and shorter slopes. Specifications for these materials should follow the recommendations in Chapter 4. Erosion control material from an Agency-approved products list could be used in lieu of a specification. If required, geosynthetic drainage composite specifications from Chapter 2 should also be included. Drainage design, required flow capacities, and geotextile filtration criteria are project specific and require specific evaluation for a project. Line and grade drawings, erosion control design, and a system specification are required for the second option. Alternatives in specified materials or systems are possible during design and post-award periods.

All feasible, cost-effective material alternates should be seriously considered for in-house design projects:

1. Alternate materials during the design phase - Consultants and the Agency should consider all **approved** materials and shall provide generic specifications for geosynthetic reinforcement and other materials used in the system.
2. Value engineering (VE) may be applied by the Contractor to the use of material alternates. Proposals incorporating unapproved materials, not submitted for review prior to a project's design phase, **should not be approved** for VE proposals.

All feasible, cost-effective material system alternates should be seriously considered with a line and grade approach:

1. Alternate systems during the design phase - Consultants and the Agency shall consider all feasible alternates and provide at least two approved reinforced slope system or design alternates (*i.e.*, bid reinforced slope against wall or flatter slope options) whenever possible.
2. Value engineering may be applied by the Contractor to design alternates. Slope system proposals, which may be designated an experimental feature, or which incorporate unapproved materials, **should not be approved** for VE proposals.

8.8-1 Specification for Geosynthetic Soil Reinforcement (Material and construction specification for an *Agency Design*) (after Berg, 1993).

1. DESCRIPTION:

Work shall consist of furnishing geosynthetic soil reinforcement for use in construction of reinforced soil slopes.

2. GEOSYNTHETIC REINFORCEMENT MATERIAL:

2.1 The specific geosynthetic reinforcement material and supplier shall be preapproved by the Agency as outlined in the Agency's reinforced slope policy.

2.2 The geosynthetic reinforcement shall consist of a geogrid or geotextile that can develop sufficient mechanical interlock with the surrounding soil or rock. The geosynthetic reinforcement structure shall be dimensionally stable and able to retain its geometry under construction stresses and shall have high resistance to damage during construction, to ultraviolet degradation, and to all forms of chemical and biological degradation encountered in the soil being reinforced.

2.3 The geosynthetics shall have an Allowable Strength (T_{al}) and Pullout Resistance, for the soil type(s) indicated, as listed in Table 8-2 for geotextiles and/or Table 8-3 for geogrids.

2.4 The permeability of a geotextile reinforcement shall be greater than the permeability of the fill soil.

2.5 Certification: The contractor shall submit a manufacturer's certification that the geosynthetics supplied meet the respective index criteria set when geosynthetic was approved by the Agency, measured in full accordance with all test methods and standards specified. In case of dispute over validity of values, the Engineer can require the Contractor to supply test data from an Agency-approved laboratory to support the certified values submitted.

2.6 Quality Assurance/Index Properties: Testing procedures for measuring design properties require elaborate equipment, tedious set up procedures and long durations for testing. These tests are inappropriate for quality assurance (QA) testing of geosynthetic reinforcements received on site. In lieu of these tests for design properties, a series of index criteria may be established for QA testing. These index criteria include mechanical and geometric properties that directly impact the design strength and soil interaction behavior of geosynthetics. **It is likely that each family of products will have varying index properties and QC/QA test methods.** QA testing should measure the respective index criteria set when geosynthetic was approved by the Agency. Minimum average roll values, per ASTM D 4759, shall be used for conformance.

3. CONSTRUCTION:

3.1 Delivery, Storage, and Handling: Follow requirements set forth under materials specifications for geosynthetic reinforcement, drainage composite, and geosynthetic erosion mat.

3.2 On-Site Representative: Geosynthetic reinforcement material suppliers shall provide a qualified and experienced representative on site, for a minimum of three days, to assist the Contractor and Agency inspectors at the start of construction. If there is more than one slope on a project then this criteria will apply to construction of the initial slope only. The representative shall also be available on an as-needed basis, as requested by the Agency Engineer, during construction of the remaining slope(s).

3.3 Site Excavation: All areas immediately beneath the installation area for the geosynthetic reinforcement shall be properly prepared as detailed on the plans, specified elsewhere within the specifications, or directed by the Engineer. Subgrade surface shall be level, free from deleterious materials, loose, or otherwise unsuitable soils. Prior to placement of geosynthetic reinforcement, subgrade shall be proofrolled to provide a uniform and firm surface. Any soft areas, as determined by the Owner's Engineer, shall be excavated and replaced with suitable compacted soils. Foundation surface shall be inspected and approved by the Owner's Geotechnical Engineer prior to fill placement. Benching the backcut into competent soil is recommended to improve stability.

3.4 Geosynthetic Placement: The geosynthetic reinforcement shall be installed in accordance with the manufacturer's recommendations. The geosynthetic reinforcement shall be placed within the layers of the compacted soil as shown on the plans or as directed.

The geosynthetic reinforcement shall be placed in continuous, longitudinal strips in the direction of main reinforcement. However, if the Contractor is unable to complete a required length with a single continuous length of geogrid, a joint may be made with the Engineer's approval. Only one joint per length of geogrid shall be allowed. This joint shall be made for the full width of the strip by using a similar material with similar strength. Joints in geogrid reinforcement shall be pulled and held taut during fill placement. Joints shall not be used with geotextiles.

Adjacent strips, in the case of 100% coverage in plan view, need not be overlapped. The minimum horizontal coverage is 50%, with horizontal spacings between reinforcement no greater than 1 m. Horizontal coverage of less than 100% shall not be allowed unless specifically detailed in the construction drawings.

Adjacent rolls of geosynthetic reinforcement shall be overlapped or mechanically connected where exposed in a wrap-around face system, as applicable.

Place only that amount of geosynthetic reinforcement required for immediately pending work to prevent undue damage. After a layer of geosynthetic reinforcement has been placed, the next succeeding layer of soil shall be placed and compacted as appropriate. After the specified soil layer has been placed, the next geosynthetic reinforcement layer shall be installed. The process shall be repeated for each subsequent layer of geosynthetic reinforcement and soil.

Geosynthetic reinforcement shall be placed to lay flat and pulled tight prior to backfilling. After a layer of geosynthetic reinforcement has been placed, suitable means, such as pins or small piles of soil, shall be used to hold the geosynthetic reinforcement in position until the subsequent soil layer can be placed. Under no circumstances shall a track-type vehicle be allowed on the geosynthetic reinforcement before at least 150 mm of soil has been placed.

During construction, the surface of the fill should be kept approximately horizontal. Geosynthetic reinforcement shall be placed directly on the compacted horizontal fill surface. Geosynthetic reinforcements are to be placed within 75 mm of the design elevations and extend the length as shown on the elevation view, unless otherwise directed by the Owner's Engineer. Correct orientation of the geosynthetic reinforcement shall be verified by the Contractor.

3.5 Fill Placement: Fill shall be compacted as specified by project specifications or to at least 95 percent of the maximum density determined in accordance with AASHTO T-99, whichever is greater.

Density testing shall be made every 500 cubic meters of soil placement or as otherwise specified by the Owner's Engineer or contract documents.

Backfill shall be placed, spread, and compacted in such a manner to minimize the development of wrinkles and/or displacement of the geosynthetic reinforcement.

Fill shall be placed in 300 mm maximum lift thickness where heavy compaction equipment is to be used, and 150 mm maximum uncompacted lift thickness where hand-operated equipment is used.

Backfill shall be graded away from the slope crest and rolled at the end of each workday to prevent ponding of water on the surface of the reinforced soil mass.

Tracked construction equipment shall not be operated directly upon the geosynthetic reinforcement. A minimum fill thickness of 150 mm is required prior to operation of tracked vehicles over the geosynthetic reinforcement. Turning of tracked vehicles should be kept to a minimum to prevent tracks from displacing the fill and the geosynthetic reinforcement.

If recommended by the geogrid manufacturer and approved by the Engineer, rubber-tired equipment may pass over the geogrid reinforcement at slow speeds, less than 10 mph. Sudden braking and sharp turning shall be avoided.

3.6 Erosion Control Material Installation: See Erosion Control Material Specification for installation notes.

3.7 Geosynthetic Drainage Composite: See Geocomposite Drainage Composite Material Specification for installation notes.

3.8 Final Slope Geometry Verification: Contractor shall confirm that as-built slope geometries conform to approximate geometries shown on construction drawings.

4. METHOD OF MEASUREMENT:

Measurement of geosynthetic reinforcement is on a square-meter basis and is computed on the total area of geosynthetic reinforcement shown on the construction drawings, exclusive of the area of geosynthetics used in any overlaps. Overlaps are an incidental item.

5. BASIS OF PAYMENT:

5.1 The accepted quantities of geosynthetic reinforcement by type will be paid for per-square-meter in-place.

5.2 Payment will be made under:

Pay Item	Pay Unit
Geogrid Soil Reinforcement - Type I	square meter
Geogrid Soil Reinforcement - Type II	square meter
Geogrid Soil Reinforcement - Type III	square meter
or	
Geotextile Soil Reinforcement - Type I	square meter
Geotextile Soil Reinforcement - Type II	square meter
Geotextile Soil Reinforcement - Type III	square meter
Material Supplier Representative (exceeding 3 days)	person-day

TABLE 8-2
ALLOWABLE GEOTEXTILE STRENGTH AND PULLOUT RESISTANCE WITH VARIOUS SOIL TYPES[1]

Geotextile	Ultimate Strength T_{ULT} (kN/m)[1] per ASTM D 4595	Allowable Strength T_{al} (kN/m)[2]	Pullout Resistance Factor[2] F*	Minimum Permeability, k[4] (or Permittivitty, ψ) per ASTM D 4491	For use with these fills[3]
A					GW-GM
A					SW-SM-SC
B					GW-GM
B					SW-SM-SC

NOTES:
1. Based upon minimum average roll values (MARV).
2. See Appendix K.
3. Unified Soil Classifications.
4. Equal to or greater than the permeability of the fill soil.

TABLE 8-3 ALLOWABLE GEOGRID STRENGTH AND PULLOUT RESISTANCE WITH VARIOUS SOIL TYPES[1]				
Geogrid	Ultimate Strength T_{ULT} (kN/m)[1] per GRI: GG1	Allowable Strength T_{al} (kN/m)[2]	Pullout Resistance Factor[2] F*	For use with these fills[3]
A				GW-GM
A				SW-SM-SC
B				GW-GM
B				SW-SM-SC
NOTES: 1. Based upon minimum average roll values (MARV). 2. See Appendix K. 3. Unified Soil Classifications.				

8.8-2 Specification for Geosynthetic Reinforced Soil Slope System (*Vendor/Contractor Design*) (after Berg, 1993)

1.0 DESCRIPTION

Work shall consist of design, furnishing materials, and construction of geosynthetic reinforced soil slope structure. Supply of geosynthetic reinforcement, drainage composite, and erosion control materials, and site assistance are all to be furnished by the slope system supplier.

2.0 REINFORCED SLOPE SYSTEM

2.1 Acceptable Suppliers — The following suppliers can provide an agency approved system:
(a) ____________________
(b) ____________________
(c) ____________________

2.2 Materials: Only those geosynthetic reinforcement, drainage composite, and erosion mat materials approved by the Agency prior to advertisement for bids on the project under consideration shall be utilized in the slope construction. Geogrid Soil Reinforcement, Geotextile Soil Reinforcement, Drainage Composite, and Geosynthetic Erosion Mat materials are specified under respective material specifications.

2.3 Design Submittal: The Contractor shall submit six (6) sets of detailed design calculations, construction drawings, and shop drawings for approval within thirty (30) days of authorization to proceed and at least sixty (60) days prior to the beginning of reinforced slope construction. The calculations and drawings shall be prepared and sealed by a professional engineer, licensed in the State. Submittal shall conform to Agency requirements for steepened reinforced soil systems.

2.4 Material Submittals: The Contractor shall submit six (6) sets of manufacturer's certification that indicate the geosynthetic soil reinforcement, drainage composite, and geosynthetic erosion mat meet the requirements set forth in the respective material specifications, for approval at least sixty (60) days prior to the start of reinforced slope construction.

3.0 CONSTRUCTION
(Should follow the specification details in Section 8.8-1)

4.0 METHOD OF MEASUREMENT
Measurement of Geosynthetic Reinforced Soil Slope Systems is on a vertical square meter basis.

Payment shall cover reinforced slope design, and supply and installation of geosynthetic soil reinforcement, reinforced soil fill, drainage composite, and geosynthetic erosion mat. Excavation of any suitable materials and replacement with select fill, as directed by the Engineer, shall be paid under a separate pay item.

Quantities of reinforced soil slope system as shown on the plans may be increased or decreased at the direction of the Engineer based on construction procedures and actual site conditions.

5.0 BASIS OF PAYMENT

5.1 The accepted quantities of geosynthetic reinforced soil slope system will be paid for per vertical square-meter in-place.

5.2 Payment will be made under:

Pay Item	Pay Unit
Geosynthetic Slope System	vertical square-meter
Material Supplier Representative	person-day

8.9 INSTALLATION PROCEDURES

Reinforcement layers are easily incorporated between the compacted lifts of fill. Therefore, construction of reinforced slopes is very similar to normal embankment construction. The following is the usual construction sequence:

A. Site preparation.
- Clear and grub site.
- Remove all slide debris (if a slope reinstatement project).
- Prepare level subgrade for placement of first level of reinforcing.
- Proofroll subgrade at the base of the slope with roller or rubber-tired vehicle.

B. Place the first reinforcing layer.
- Reinforcement should be placed with the principal strength direction perpendicular to the face of slope.
- Secure reinforcement with retaining pins to prevent movement during fill placement.
- A minimum overlap of 150 mm is recommended along the edges perpendicular to the slope for wrapped-face structures. Alternatively, with geogrid reinforcement, the edges may be clipped or tied together. When geosynthetics are not required for face support, no overlap is required and edges should be butted.

C. Place backfill on reinforcement.

- Place fill to required lift thickness on the reinforcement using a front-end loader operating on previously placed fill or natural ground.
- Maintain a minimum of 150 mm between reinforcement and wheels of construction equipment. This requirement may be waived for rubber-tired equipment provided that field trials, including geosynthetic strength tests, have demonstrated that anticipated traffic conditions will not damage the specific geosynthetic reinforcement.
- Compact with a vibratory roller or plate-type compactor for granular materials, or a rubber-tired vehicle for cohesive materials.
- When placing and compacting the backfill material, avoid any deformation or movement of the reinforcement.
- Use lightweight compaction equipment near the slope face to help maintain face alignment.

D. Compaction control.

- Provide close control on the water content and backfill density. It should be compacted at least 95% of the standard AASHTO T99 or ASTM D 698 maximum density within 2% of optimum moisture.
- If the backfill is a coarse aggregate, then a relative density or a method type compaction specification should be used.

E. Face construction.

As indicated in the design section (8.3-3), a face wrap generally is not required for slopes up to 1H:1V, if the reinforcement is maintained at close spacing (*i.e.*, every lift or every other lift, but no greater than 400 mm). In this case the reinforcement can be simply extended to the face. For this option, a facing treatment should be applied to prevent erosion during and after construction. If slope facing is required to prevent sloughing (*i.e.*, slope angle β is greater than ϕ_{soil}) or erosion, sufficient reinforcement lengths could be provided for a wrapped-face structure. The following procedures are recommended for wrapping the face.

- Turn up reinforcement at the face of the slope and return the reinforcement a minimum of 1 to 1.2 m into the embankment below the next reinforcement layer (see Figure 8-9).
- For steep slopes, form work may be required to support the face during construction, especially if lift thicknesses of (0.5 to 0.6 m or greater) are used.
- For geogrids, a fine mesh screen or geotextile may be required at the face to retain backfill materials.

F. Continue with additional reinforcing materials and backfill.

NOTE: If drainage layers are required, they should be constructed directly behind or on the sides of the reinforced section.

Several construction photos from reinforced slope projects are shown in Figure 8-10.

8.10 FIELD INSPECTION

As with all geosynthetic construction, and especially with critical structures such as reinforced slopes, competent and professional field inspection is absolutely essential for successful construction. Field personnel must be properly trained to observe every phase of the construction. They must make sure that the specified material is delivered to the project, that the geosynthetic is not damaged during construction, and that the specified sequence of construction operations are explicitly followed. Field personnel should review the checklist items in Section 1.7. Other important details include construction of the slope face and application of the facing treatment to minimize geosynthetic exposure to ultraviolet light.

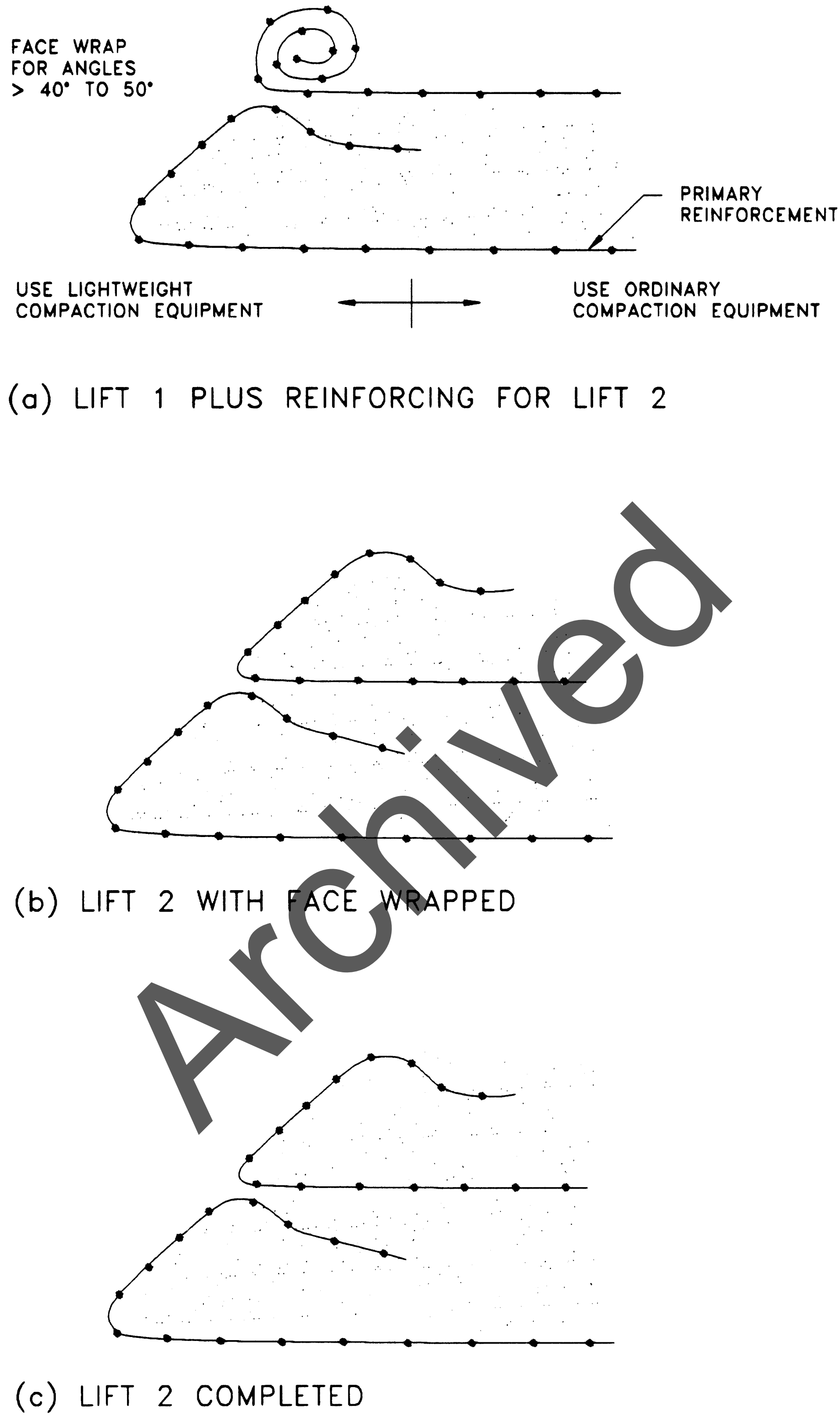
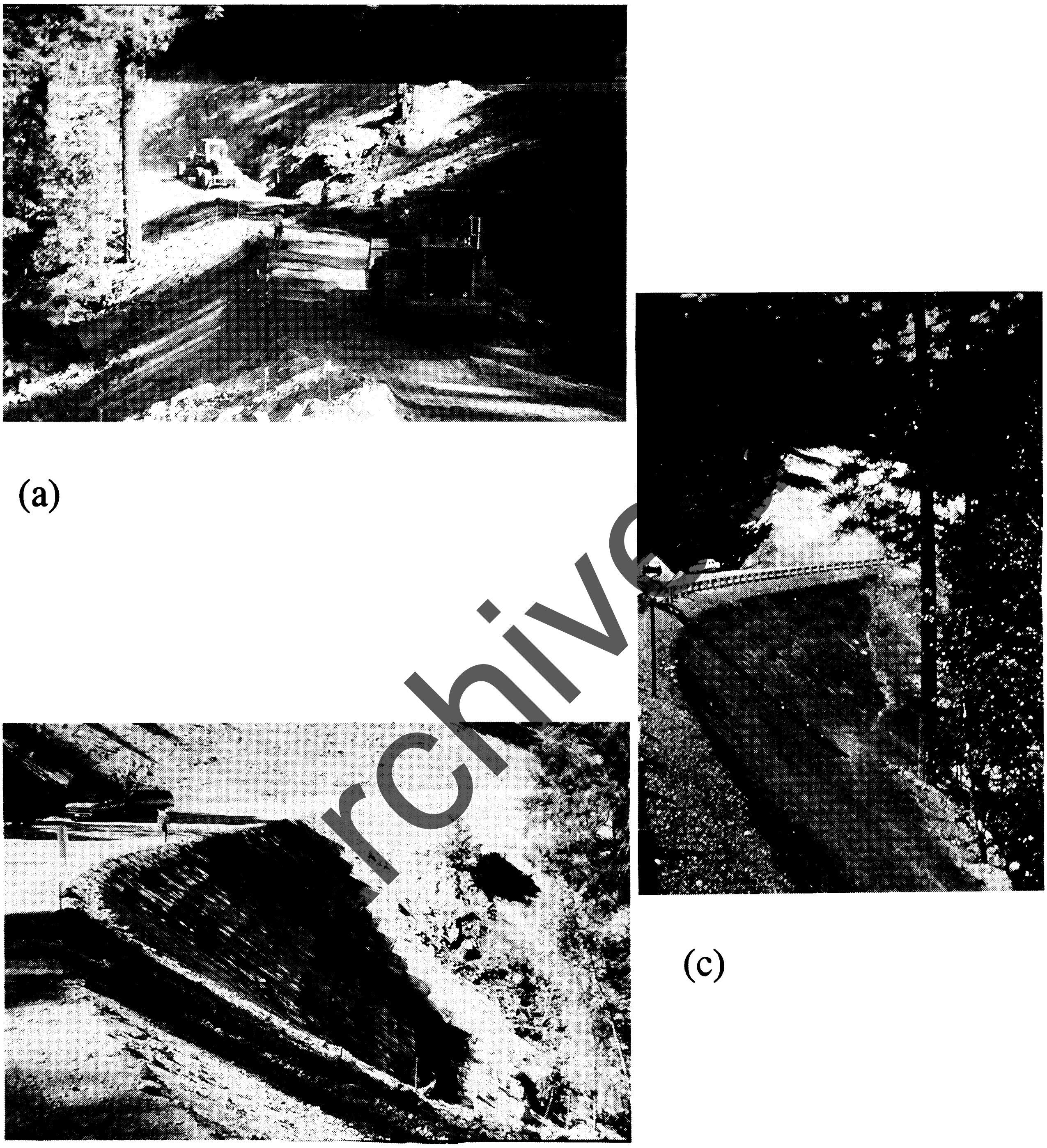


Figure 8-9 Construction of reinforced slopes.



(a)

(c)

(b)

Figure 8-10 Reinforced slope construction: a) geogrid and fill placement; b) soil fill erosion control mat facing; and c) finished, vegetated 1:1 slope.

8.11 REFERENCES

References quoted within this section are listed below. The FHWA-SA-96-071 manual (Elias and Christopher, 1997) reference is a recent, comprehensive guideline specifically addressing reinforced slopes in transportation applications. It is a *key* reference for design, specification, and contracting. This and other *key* references are noted in bold type.

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9.0 MECHANICALLY STABILIZED EARTH RETAINING WALLS AND ABUTMENTS

The purpose of this chapter is to review the variety of available geosynthetic MSE wall types, discuss their typical use, consider the advantages of geosynthetic MSE walls, and detail how they are designed, specified, and constructed. Additional design details are provided in MSE Walls and Reinforced Soil Slopes Demo 82 participants manual (Elias and Christopher, 1997); the most recent edition of AASHTO Standard Specification for Bridges (1996, with 1997 interims); and, the Design and Construction Guidelines for Reinforced Soil Structures (Christopher, et al., 1989).

9.1 BACKGROUND

Retaining walls in transportation engineering are quite common. They are required where a slope is uneconomical or not technically feasible. When selecting a retaining wall type, mechanically stabilized earth (MSE) walls should always be considered. MSE (*i.e.*, reinforced soil) walls are basically comprised of some type of reinforcing element in the soil fill to help resist lateral earth pressures. When compared with conventional retaining wall systems, there are often significant advantages to MSE retaining walls. MSE walls are very cost effective, especially for walls in fill embankment cross sections. Furthermore, these systems are more flexible than conventional earth retaining walls such as reinforced concrete cantilever or gravity walls. Therefore, they are very suitable for sites with poor foundations and for seismically active areas.

The modern invention of reinforcing the soil in fill applications was developed by Vidal in France in the mid-1960s. The Vidal system, called Reinforced Earth™, uses metal strips for reinforcement, as shown schematically in Figure 9-1. The design and construction of Reinforced Earth™ walls is quite well established, and thousands have been successfully built worldwide in the last 25 years. During this time, other similar reinforcing systems, both proprietary and nonproprietary, utilizing different types of metallic reinforcement have been developed (*e.g.*, VSL, Hilfiker, etc.; see Mitchell and Villet, 1987, and Christopher, et al., 1989).

The use of geogrids or geotextiles rather than metallic strips (ties), shown conceptually in Figure 9-2, is really a further development of the Reinforced Earth™ concept. Geosynthetics offer a viable and often very economical alternative to metallic reinforcement for both permanent and temporary walls, especially under certain environmental conditions. Reinforcing with geosynthetics has been used since 1977 (Bell et al., 1975). Today the use in transportation walls is quite common in many states. The maximum heights of geosynthetic reinforced walls constructed to date are about 15 m, whereas metallic reinforced walls have exceeded 30 m in

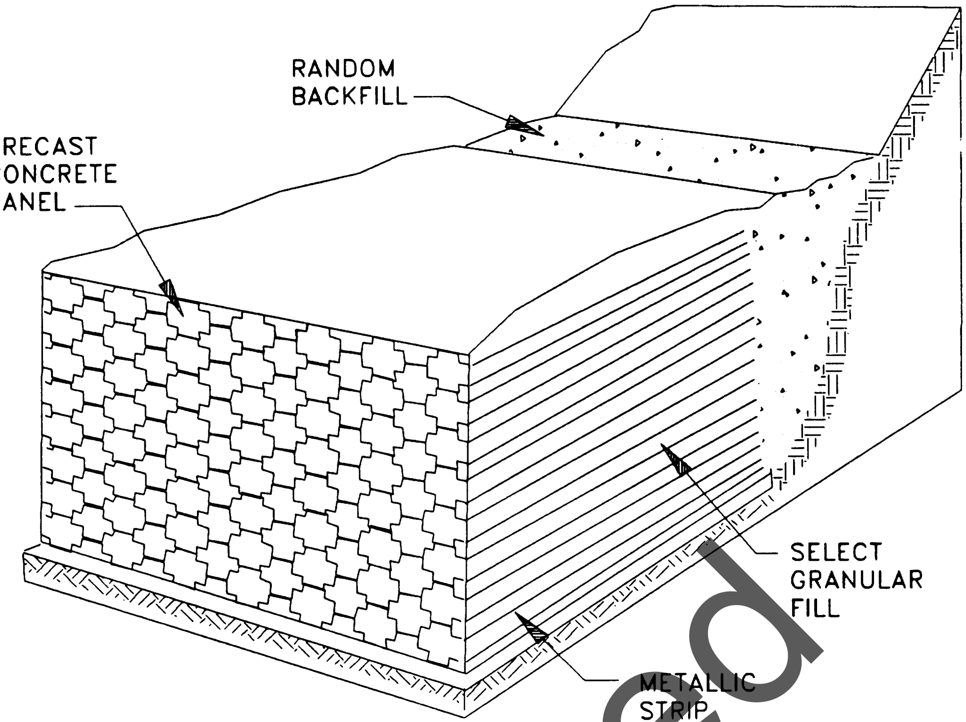


Figure 9-1 Component parts of a Reinforced Earth wall (Lee, Adams, and Vagneron, 1973).

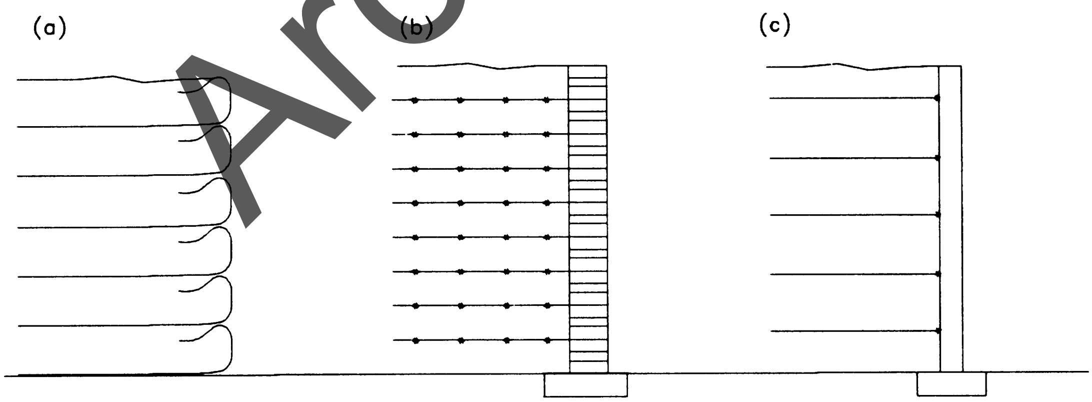


Figure 9-2 Reinforced retaining wall systems using geosynthetics: (a) with *wrap-around* geosynthetic facing, (b) with segmented precast concrete or timber panels, and (c) with full-height (*propped*) precast panels.

height. A significant benefit of using geosynthetics is the wide variety of wall facings available, resulting in greater aesthetic options. Metallic reinforcement is typically used with articulated precast concrete panels. Alternate facing systems for geosynthetic reinforced walls are discussed in Section 9.3.

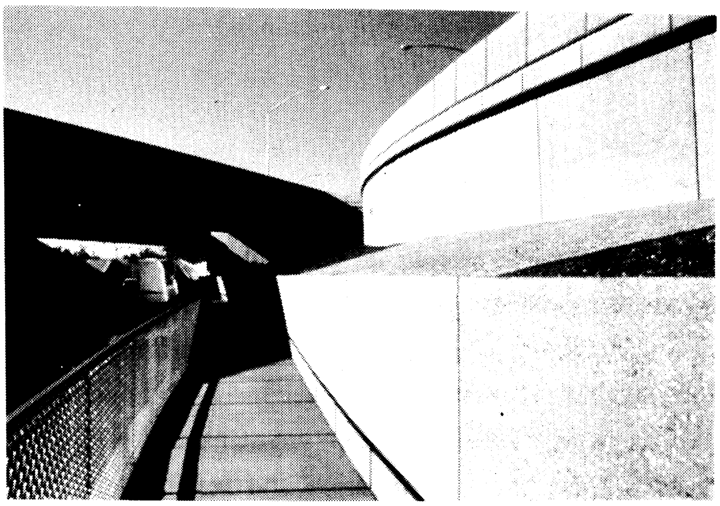
9.2 APPLICATIONS

MSE structures, including those reinforced with geosynthetics, should be considered as cost-effective alternates for all applications where conventional gravity, cast-in-place concrete cantilever, bin-type, or metallic reinforced soil retaining walls are specified. This includes bridge abutments as well as locations where conventional earthen embankments cannot be constructed due to right-of-way restrictions (another alternative is a reinforced slope, see Chapter 8). Conceptually, geosynthetic MSE walls can be used for any fill wall situation and for low- to moderate-height cut-wall situations. Similar to other MSE wall types, the relatively wide wall base width required for geosynthetic walls typically precludes their use in tall cut situations. Figure 9-3 shows several completed geosynthetic reinforced retaining walls.

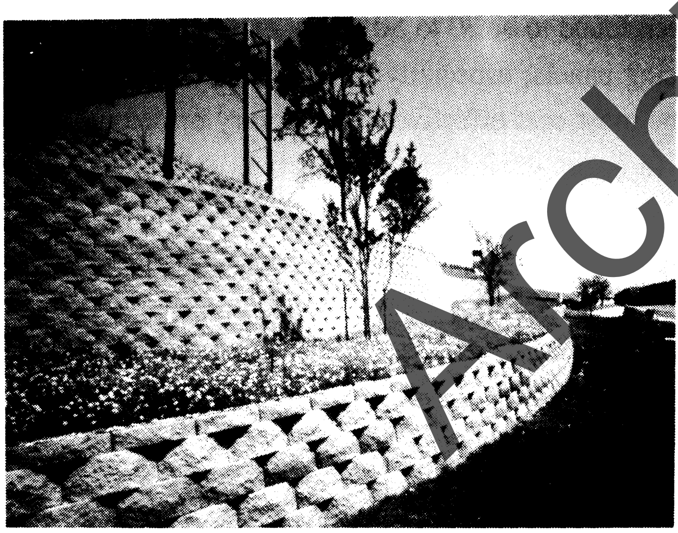
Geosynthetic MSE walls are generally less expensive than conventional earth retaining systems. Using geogrids or geotextiles as reinforcement has been found to be 30 to 50% less expensive than other reinforced soil construction with concrete facing panels, especially for small- to medium-sized projects (Allen and Holtz, 1991). They may be most cost-effective in temporary or detour construction, and in low-volume road construction (*e.g.*, national forests and parks).

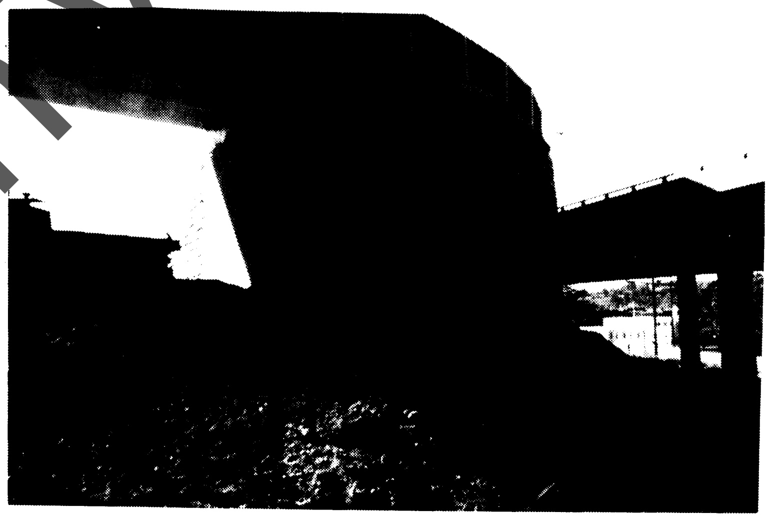
Due to their greater flexibility, MSE walls offer significant technical and cost advantages over conventional gravity or reinforced concrete cantilever walls at sites with poor foundations and/or slope conditions. These sites commonly require costly additional construction procedures, such as: deep foundations, excavation and replacement, or other foundation soil improvement techniques.

The level of confidence needed for the design of a geosynthetic wall depends on the criticality of the project (Carroll and Richardson, 1986). The criticality depends on the design life, maximum wall height, the soil environment, risk of loss of life, and impact to the public and to other structures if failure occurs. Assessment of criticality is rather subjective, and sound engineering judgement is required. The Engineer or regulatory authority should determine the critical nature of a given application. **Design, as summarized and discussed within this chapter, assumes that walls are classified as permanent, critical structures.** Of course, the method could be conservatively used to design temporary and other non-critical structures.



(a) (b)





(c) (d)

Figure 9-3 Examples of geosynthetic MSE walls: a. full-height panels, geogrid-reinforced walls, Arizona; b. full-height panels, geogrid-reinforced walls, Utah; c. modular block wall units, geogrid-reinforced walls, Texas; and d. temporary geotextile wrap around wall, Washington.

9.3 DESCRIPTION OF MSE WALLS

9.3-1 Soil Reinforcement

Geosynthetic MSE walls may utilize geogrids or geotextiles as soil reinforcing elements. However, the prevalent material used in highway walls today is geogrid reinforcement. This trend is driven both by needs of transportation agencies and by geosynthetic manufacturers and suppliers of packaged wall systems. One of these needs --enhanced aesthetics of the completed wall -- is obviously controlled by the facing used. As discussed below, the facing can dictate a preferred type of geosynthetic reinforcement.

9.3-2 Facings

A significant advantage of geosynthetic MSE walls over other earth retaining structures is the variety of facings that can be used and the resulting aesthetic options that can be provided. Descriptions of various facings are provided below. Some examples are illustrated in Figure 9-4.

Modular Block Wall (MBW) Units are the most common facing currently used for geosynthetic MSE wall construction. These facing elements are also known as modular block wall units. They are popular because of their aesthetic appeal, widespread availability, and relative low cost (Berg, 1991). A broken block, or natural stone-like, finish is the most popular MBW unit face finish.

MBW units are relatively small, squat concrete units, specially designed and manufactured for retaining wall applications. The units are typically manufactured by a dry casting process and weigh 15 to 50 kg each, with 35 to 50 kg units routinely used for highway works. The nominal depth (dimension perpendicular to wall face) of MBW units usually ranges between 0.2 and 0.6 m. The depth dimension is so great that labeling these units as MSE wall *facings* is a misnomer. They can provide significant contribution to stability, particularly for low- to moderate-height walls. Unit heights typically vary between 100 and 200 mm for the various manufacturers. Exposed face length typically ranges between 200 and 600 mm.

MBW units may be manufactured solid or with cores. Full height cores are typically filled with aggregate during erection. Units are normally dry-stacked (*i.e.*, without mortar) in a running bond configuration. Vertically adjacent units may be connected with shear pins or lips.

The vertical connection mechanism between MBW units also contributes to the connection strength between the geosynthetic reinforcement and the MBW units. Connection strength must be addressed in design, and can control the maximum allowable tensile load in a given layer of reinforcement. Therefore, the reinforcement design strength and vertical spacing of layers is specific to the particular combination of MBW unit and geosynthetic reinforcement utilized.

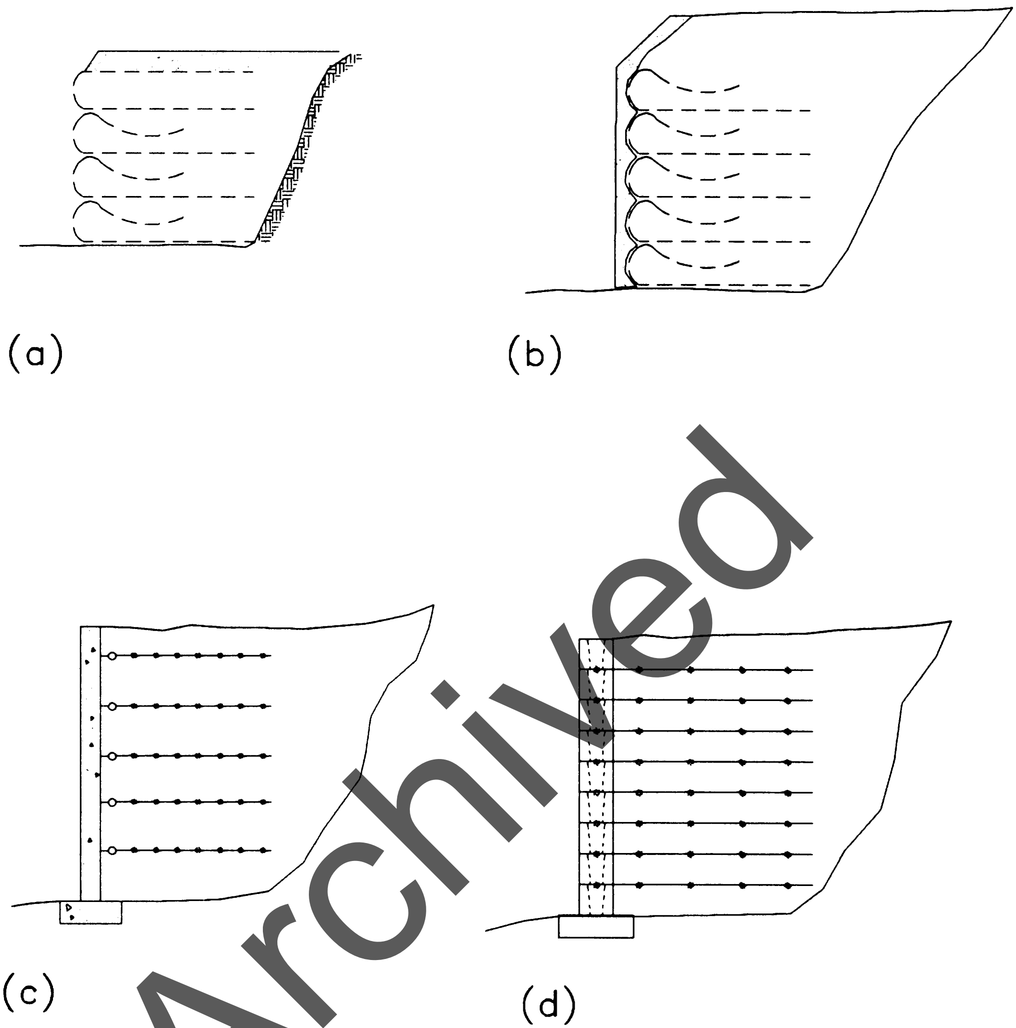


Figure 9-4 Possible geosynthetic MSE wall facings: (a) geosynthetic facing - temporary wall; (b) geosynthetic facing protected by shotcrete; (c) full-height precast concrete (propped) panels; and (d) modular concrete units.

Geogrids, both stiff and flexible, are the common reinforcing elements of MBW unit-faced MSE walls in highway applications. Geotextiles have been used in walls with MBW unit facings, but to a limited extent. A detailed description of MBW units, and design with these units, is presented by Simac et al. (1993) and is summarized by Bathurst and Simac (1994).

Wrap-Around facings are commonly used: i) for temporary structures; ii) for walls that will be subject to significant post-construction settlement; iii) where aesthetic requirements are low; and iv) where post-construction facings are applied for protection and aesthetics. The geosynthetic facing may be left exposed for temporary walls if the geosynthetic is stabilized against ultraviolet light degradation. A consistent vertical spacing of reinforcement, and therefore wrap height, of 0.3 to 0.5 m is typically used. A sprayed concrete facing is usually applied to permanent walls to provide protection against ultraviolet exposure, potential vandalism, and possible fire. Precast concrete or wood panels may also be attached after construction.

Geotextiles are commonly used in wrap-around-faced MSE walls. With the proper ultraviolet light stabilizer, these structures perform satisfactorily for a few years. They should be covered by a permanent facing for longer-term applications. Geogrids are also used for wrap facings, though a geotextile, an erosion control blanket, or sod is required to retain fill soil. Alternatively, rock or gravel can be used in the wrap area and a filter placed between the stone and fill soil. Secondary, biaxial geogrid can be used to provide the face wrap for the primary, uniaxial soil reinforcing elements for geogrids. With the proper ultraviolet light stabilizer specified, geogrids can be left uncovered for a number of years; reportedly for design lives of 50 years or more for heavy, stiff geogrids (Wrigley, 1987).

Segmental precast concrete panels, similar to panels used to face metallic MSE systems, have been used to face geosynthetic reinforced MSE walls. However, this facing is currently used in only a few states.

Stiff, polyethylene geogrids are almost exclusively used for precast concrete panel faces, where tabs of the geogrid are cast into the concrete and field-attached to the soil reinforcing geogrid layers. Flexible, polyester geogrids are not used because casting them into wet concrete would expose the geogrids to a high-alkaline environment.

Full-height concrete panels are also used to face geosynthetic MSE walls. These are used in only a few states, and only where aesthetics of full-height panels are specifically desired. Similar to the segmental precast concrete panels, stiff, polyethylene geogrids are exclusively used for precast concrete panel faces where tabs are cast into the panel.

Timber facings are commonly used for geosynthetic MSE walls. Timber-faced walls are normally used for low- to moderate-height structures, landscaping, or maintenance construction. Geotextiles and geogrids are used with timber facings.

Gabion facings are also used for MSE wall structures. Geogrid or geotextile reinforcing elements can be used. A geotextile filter is typically used between the back face of the gabion baskets and the fill soil to prevent soil from piping through the gabion stones.

9.4 DESIGN GUIDELINES FOR MSE WALLS

9.4-1 Approaches and Models

A number of approaches to geotextile and geogrid reinforced retaining wall design have been proposed, and these are summarized by Christopher and Holtz (1985), Mitchell and Villet (1987), Christopher, et al. (1989), and Claybourn and Wu (1993). The most commonly used method is classical Rankine earth pressure theory combined with tensile-resistant *tie-backs*, in which the reinforcement extends beyond an assumed Rankine failure plane. Figure 9-5 shows a MBW unit-faced system and the model typically analyzed. Because this design approach was first proposed by Steward, Williamson, and Mohney (1977) of the U.S. Forest Service, it is often referred to as the *Forest Service* or *tie-back wedge* method.

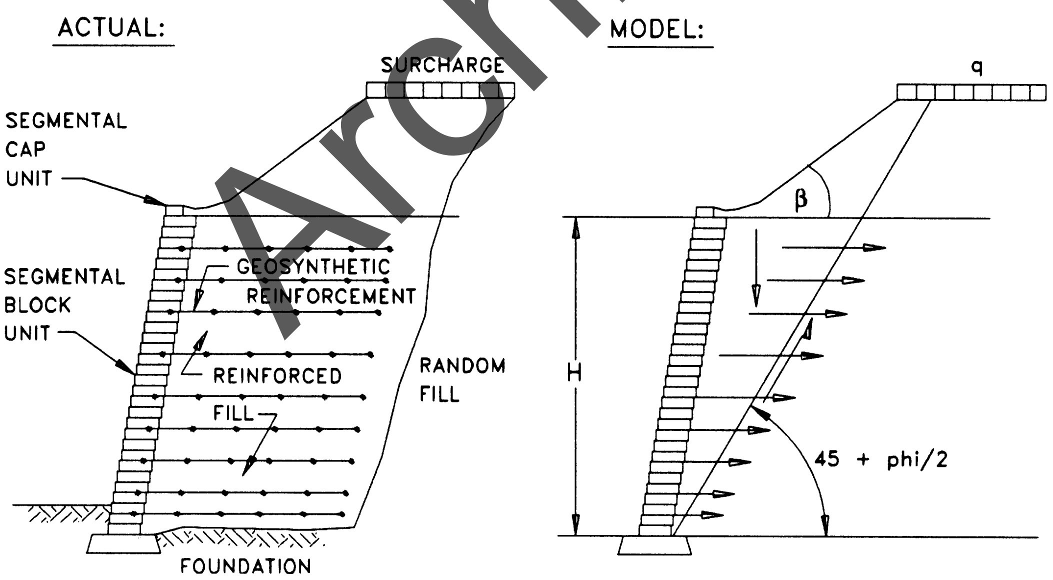


Figure 9-5 Actual geosynthetic reinforced soil wall in contrast to the design model.

The *simplified coherent gravity* method (Elias and Christopher, 1997 and AASHTO, 1996, with 1997 interims) is recommended for internal design of MSE walls for transportation works. This simplified approach was developed so that iterative design procedures are avoided and by practical considerations of some of the complex refinements of the available methods (*i.e.*, the coherent gravity method (AASHTO, 1996, with 1997 interims) and the structural stiffness method (Christopher, et al., 1990). For geosynthetics, it is essentially the same as the tie-back wedge method with a few minor changes as noted in the following paragraphs.

The simplified method uses an assumed Rankine failure surface. The lateral earth pressure coefficient, K, is determined by applying a multiplier to the active earth pressure coefficient. The active earth pressure coefficient is determined using a Coulomb earth pressure relationship, assuming no wall friction and a horizontal backslope (β angle equal to zero). For a vertical wall the earth pressure, therefore, reduces to the Rankine active earth pressure equation. For wall face batters in excess of 10 degrees, a simplified form of the Coulomb equation can be used, as discussed in section 9.4.3.

The basic approach for internal stability is a limiting equilibrium analysis, with consideration of the reinforced soil mass's possible failure modes as given in Table 9-1. These failure modes are analogous to those of metallic reinforced MSE walls.

As with conventional retaining structures, overall (external) stability and wall settlement must also be satisfactory. In fact, external stability considerations (*i.e.*, sliding) generally control the length of the reinforcement required.

TABLE 9-1
FAILURE MODES AND REQUIRED PROPERTIES FOR MSE WALLS

Failure Mode for Geosynthetic Reinforcement	Failure Mode for Metallic Reinforcement	Property Required
Geogrid or geotextile rupture	Strips or meshes break	Tensile strength
Geogrid or geotextile pullout	Strips or meshes pullout	Soil-reinforcement interaction (passive resistance, frictional resistance)
Excessive creep of geogrid or geotextile	N/A	Creep resistance
Connection Failure	Strips or meshes break, or strips or meshes pullout	Tensile failure or pullout

9.4-2 Design Steps

The following is a step-by-step procedure for the design of geosynthetic reinforced walls.

STEP 1. Establish design limits, scope of project, and external loads (Figure 9-6).

A. Wall height, H

B. Wall length, L

C. Face batter angle

D. External loads:
   1. Temporary concentrated live loads, Δq
   2. Uniform surcharge loads, q
   3. Seismic loads, A_m

E. Type of facing and connections:
   1. Segmental concrete units, timbers, incremental height precast panels, etc.
   2. Full-height concrete panels
   3. Wrapped

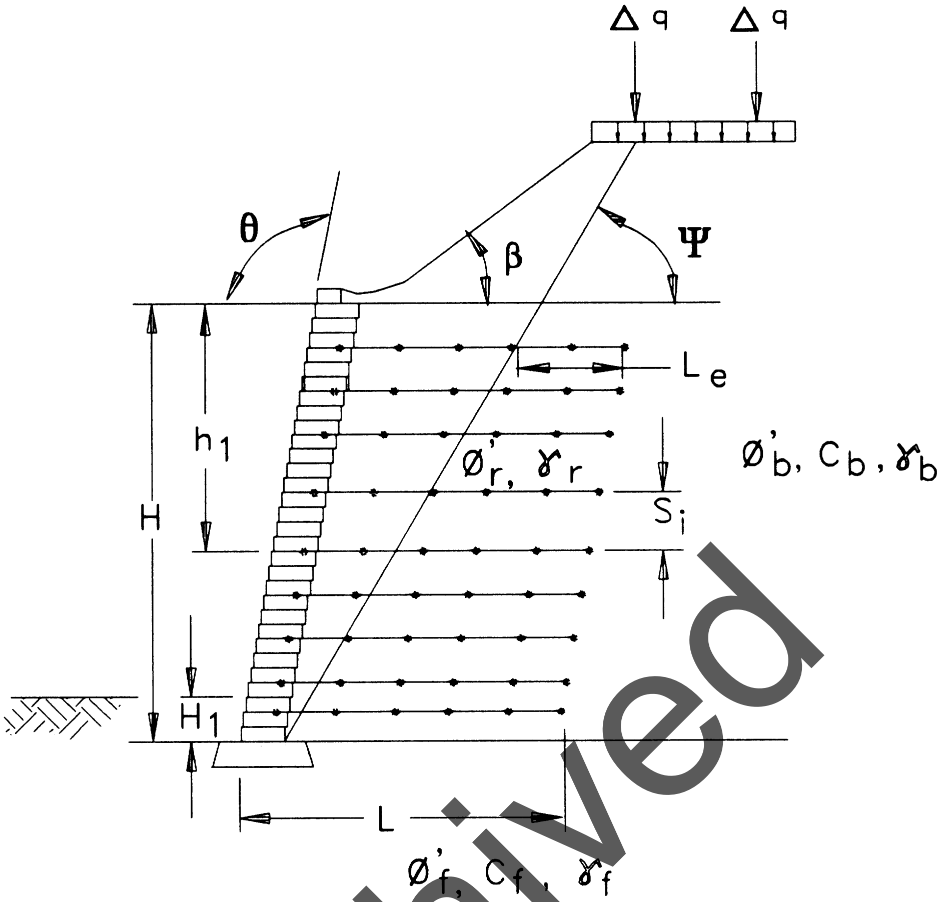
F. Spacing requirements, S_i, (if any) based on facing connections, stability during construction, lift thickness, and placement considerations (*e.g.*, maximum s = 0.5 m for geotextile- and geogrid-wrapped faced walls), and reinforcement strength.

G. Environmental conditions such as frost action, scour, shrinkage and swelling, drainage, seepage, rainfall runoff, chemical nature of backfill and seepage water (*e.g.*, pH range, hydrolysis potential, chlorides, sulfates, chemical solvents, diesel fuel, other hydrocarbons, etc.), etc.

H. Design and service life periods

STEP 2. Determine engineering properties of foundation soil (Figure 9-6).

A. Determine the soil profile below the base of the wall; exploration depth should be at least twice the height of the wall or to refusal. Borings should be spaced at least every 30 to 45 m along the wall's alignment at the front and back of the reinforced soil section.



For vertical walls

$$\psi = 45 + \frac{\phi_r'}{2}$$

For walls with face batter of 10° or more from the vertical,

$$\tan(\psi-\phi) = \frac{-\tan(\phi-\beta) + \sqrt{\tan(\phi-\beta)\,[\tan(\phi-\beta) + \cot(\phi+\theta-90)][1+\tan(\delta+90-\theta)\cot(\phi+\theta-90)]}}{1+\tan(\delta+90-\theta)[\tan(\phi-\beta)+\cot(\phi+\theta-90)]}$$

where $\delta = \beta$

Figure 9-6 Geometric and loading characteristics of geosynthetic MSE walls.

B. Determine the foundation soil strength parameters (c_u, ϕ_u, c', and ϕ'), unit weight (γ), and consolidation parameters (C_c, C_r, c_v and σ'_p) for each foundation stratum.

C. Establish location of groundwater table. Check need for drainage behind and beneath the wall.

STEP 3. Determine properties of both the reinforced fill and retained backfill soils (see Section 9.6 for recommended soil fill requirements).

A. Water content, gradation and plasticity. Note that soils with appreciable fines (silts and clays) are *not* recommended for MSE walls.

B. Compaction characteristics (maximum dry unit weight, γ_d, and optimum water content, w_{opt}, or relative density).

C. Angle of internal friction, ϕ_r'.

D. pH, oxidation agents, etc. (For a discussion of chemical and biological characteristics of the backfill that could affect geosynthetic durability, see Section 9.6.)

STEP 4. Establish design factors of safety (the values below are recommended minimums; local codes may require greater values) and performance criteria.

A. External stability:
1. Sliding: FS $\geq$ 1.5
2. Bearing capacity: FS $\geq$ 2.5
3. Deep-seated (overall) stability: FS $\geq$ 1.3
4. Settlement: Maximum allowable total and differential based on performance requirements of the project.
5. Dynamic loading: FS $\geq$ 1.1 or greater, depending on local codes.

B. Internal stability:
1. Determine the allowable long-term tensile strength, T_{al}, of the reinforcement; see Appendix K. Remember to consider connection strength between the reinforcement and facing element, which may limit the reinforcement's design tensile strength.
2. Determine the long-term design strength, T_d, of the reinforcement, where:

$$T_a = T_{al} / FS$$

with a minimum FS against internal stability failure of 1.5 normally used.

3. Pullout resistance: FS ≥ 1.5; minimum embedment length is approximately 1 m. Use FS $\geq$ 1.1 for seismic pullout.

STEP 5. Determine preliminary wall dimensions.

A. For analyzing a first trial section, assume a reinforced section length of L = 0.7H.

B. Determine wall embedment depth.

1. Minimum embedment depth, H_1, at the front of the wall (Figure 9-6):

Slope in Front of Wall	Minimum H_1
horizontal (walls)	H/20
horizontal (abutments)	H/10
3H:1V	H/10
2H:1V	H/7
3H:2V	H/5

A minimum horizontal bench of 1.2 m wide should be provided in front of walls founded on slopes.

2. Consider possible frost action, shrinkage, and swelling potential of foundation soils, global stability, and seismic activity. In any case, the minimum H_1 is 0.5 m.

STEP 6. Develop the internal and external lateral earth pressure diagrams for the reinforced section. It is recommended that computations for external stability be made assuming the reinforced soil mass and facing to be a rigid body, and for internal stability using the Simplified method (Elias and Christopher, 1997 and AASHTO, 1996, with 1997 interims).

A. Consider the internal stresses from the reinforced soil fill and dead load and live load surcharges.

B. Consider the external stresses from the retained backfill plus dead load and live load surcharges.

C. Combine earth, surcharge, and live load pressure diagrams into a composite diagram for the internal and external design.

STEP 7. Check external wall stability.

A. Sliding resistance. Check with and without surcharge.

B. Bearing capacity of the foundation.

C. Deep-seated (overall) stability.

D. Seismic analysis.

STEP 8. Estimate settlement of the reinforced section using conventional settlement analyses.

STEP 9. Calculate the maximum horizontal stress at each level of reinforcement.

A. Determine, at each reinforcement level, the vertical stress distribution due to reinforced fill weight and the uniform surcharge and resultant external forces.

B. Determine, at each reinforcement level, the additional vertical stress due to any concentrated surcharges.

C. Calculate the horizontal stresses, σ_h, using the lateral earth pressure diagram from Step 6.

STEP 10. Check internal stability and determine reinforcement requirements.

Use the lateral earth pressure diagrams developed in Step 6 for the reinforced section.

A. Calculate the maximum tension T_{max} in each reinforcement layer per unit width of wall based on the vertical spacing, S_v (Figure 9-6) of reinforcing layers to resist the internal lateral pressures.

B. Determine the long-term design strength $T_a \geq T_{max}$, where T_a is equal to the long-term allowable strength T_{al}, defined in Appendix K, divided by the FS against internal failure selected in Step 4.

C. Ensure that the connection strength of the reinforcement to the wall facing (e.g., MBW unit, timber, etc.) does not control the magnitude of reinforcement strength, T_a, that can be mobilized. Also, check the local stability of MBW units, timber, or concrete panels that are used for the wall facing. If a wrap- around face is used, determine overlap length, L_o, for the folded portion of the geosynthetic at the face.

D. Check length of the reinforcement, L_e, required to develop pullout resistance beyond the Rankine failure wedge. A minimum L_e = 1 m is recommended.

STEP 11. Prepare plans and specifications.

9.4-3 Comments on the Design Procedure

Again, for additional design details refer to the Mechanically Stabilized Earth Walls and Reinforced Soil Slopes Design and Construction Guidelines (Elias and Christopher, 1997) and to the AASHTO Standard Specification for Bridges (1996, with 1997 interims), and the Design and Construction Guidelines for Reinforced Soil Structures (Christopher, et al., 1989).

STEPS 1 and 2 need no elaboration.

STEP 3. Determine reinforced fill and retained backfill properties.

Requirements for reinforced fill are presented in Section 9.6-1. There are no specific requirements for the retained backfill soils stated in either the AASHTO or FHWA guidelines. However, retained backfill soils generally should meet state agency embankment fill soil requirements. The engineering properties of these two separate fill materials has a significant influence on the design of the reinforced soil volume.

The moist unit weights, γ_m, of the reinforced fill and retained backfill soils can be determined from the standard Proctor test (AASHTO T-99) or alternatively, from a vibratory-type relative density test. The angles of internal friction, ϕ', should be consistent with the respective design value of unit weight. Conservative estimates can be made for granular materials, or alternatively for major projects, this soil property can be determined by drained direct shear (ASTM D 3080) or triaxial tests.

Conventional compaction control density measurements should be performed for fills where a majority of the material passes a 20 mm sieve. For coarse, gravelly backfills, use either relative density for compaction control or a method-type compaction specification for fill placement. The latter is appropriate if the fill contains more than 30% of 20 mm or larger materials.

STEP 4. Establish design factors of safety.

Minimum recommended factors of safety for the various potential internal, external, and global failures are based on AASHTO and FHWA guidelines. Determine if **minimum** or higher values should be used.

STEP 5. Determine preliminary wall dimensions, including wall embedment depth.

Since the design process is trial and error, it is necessary to initially analyze a set of assumed trial wall dimensions. The recommended minimum value of $L \approx 0.7H$ is a good place to start. Surcharges and sloping fills will likely increase the reinforcement length requirements. For low ($H < 3$ m) walls, you may wish to use a minimum of $L = 2$ m, although AASHTO recommends a minimum length of 2.4 m. In any case, be sure the overall stability (Step 7) is satisfactory.

Unless the foundation is on rock, a minimum embedment depth is required to provide adequate bearing capacity and to provide for environmental considerations such as frost action, shrinkage and swelling clays, or earthquakes. The recommendations given earlier under Step 5 are conservative. Frost or moisture sensitive soils could always be removed and replaced to reduce embedment requirements.

Embedment of the wall also helps resist the lateral earth pressure exerted by the reinforced fill through passive resistance at the toe. This resistance is neglected for design purposes because it may not always be there. Construction sequence, possible scour, or future excavation at the front of the wall may eliminate it.

STEP 6. Develop lateral earth pressure diagrams for both the reinforced fill and retained backfill.

A. Determine the internal lateral stresses σ_H at any level z from the weight of the reinforced fill γz using the properties as determined in Step 3, plus any uniform surcharges q, concentrated surcharges $\Delta\sigma_v$, live loads Δq or any concentrated stresses from horizontal surcharges $\Delta\sigma_h$.

$$\sigma_H = K \sigma_v + \Delta\sigma_h$$

where: $K = K_a$ for geosynthetics; and

$$\sigma_v = \gamma_r z + q + \Delta q + \Delta\sigma_v$$

Various approaches for considering the lateral earth pressures due to distributed surcharges, concentrated surcharges, and live loads are discussed by Christopher and Holtz (1985), Christopher, et al. (1989), and Elias and Christopher (1997). Terzaghi and Peck (1967), Wu (1975), Perloff and Baron (1976), the U.S. Forest Service (Steward, et al., 1977), Simac et al. (1993), and the U.S. Navy DM-7 (1982) also provide suitable methods.

The increment of vertical stress due to concentrated vertical loads $\Delta\sigma_v$ is evaluated using a 2V:1H pyramidal distribution by AASHTO (1996, with 1997 interims) and FHWA (Elias and Christopher, 1997).

For calculating the vertical stress in the reinforced section of walls supporting a backslope on angle β, use:

$$\sigma_v = \gamma_r z + 0.5 L (\tan \beta) \gamma_r$$

For internal stability, the active earth pressure coefficient, K_a, should be determined using the Coulomb method, but assuming no wall friction and that the backslope angle, β, is always equal to zero. Thus, for a near-vertical (*e.g.*, $\leq 10°$) face batter, the Coulomb equation simplifies mathematically to the simplest form of the Rankine equation:

$$K_a = \tan^2 (45° - \phi/2) \qquad [9\text{-}1]$$

For walls with face batter $\geq 10°$ the following simplified form of the Coulomb equation could be used:

$$K_a = \frac{\sin^2(\theta + \phi)}{\sin^3\theta \left[1 + \frac{\sin \phi}{\sin \theta}\right]^2} \qquad [9\text{-}2]$$

where: θ is the face inclination clockwise from the horizontal (see Figure 9-6).

Determine the appropriate lateral earth pressure distribution diagram for the design height of the retaining wall.

In conventional retaining wall design, active earth pressure conditions (earth pressure coefficient = K_a) are normally assumed. There may be some situations, however, where the wall is prevented from moving (examples include abutments of rigid frame bridges;

walls on bedrock), and *at rest* earth pressure conditions (K_o), or even greater pressures due to compaction, are appropriate.

K_o may be estimated from the Jaky (1948) relationship:

$$K_o = 1 - \sin \phi_{cv} \qquad [9\text{-}3]$$

where ϕ_{cv} = constant volume friction angle.

B. Consider the external lateral stresses from the retained fill plus any distributed, concentrated surcharges, or live loads.

Using the retained backfill properties as determined in Step 3, calculate the lateral earth pressure coefficient and develop the external stability lateral earth pressure diagram for the wall. This pressure acts along the height, measured from bottom of wall to top of finished grade, at a vertical line at the back of the reinforced soil mass.

The lateral earth pressure coefficient, K_a, for external stability may be computed with the Rankine earth pressure equation (9-1) if the backslope angle, β, is equal to zero and the face batter is near-vertical (*e.g.*, $\leq$ 10°). The following equation should be used for sloping fill surcharges on walls with near-vertical (*e.g.*, $\leq$ 10°) face batters:

$$K_a = \cos\beta \frac{\cos\beta - \sqrt{\cos^2\beta - \cos^2\phi}}{\cos\beta + \sqrt{\cos^2\beta - \cos^2\phi}} \qquad [9\text{-}4]$$

See AASHTO or FHWA guidelines for broken back slope surcharge conditions. For face batters of 10°, or more, the coefficient of earth pressure for external stability can be calculated with the general Coulomb case as:

$$K_a = \frac{\sin^2(\theta + \phi)}{\sin^2\theta \, \sin(\theta - \delta) \left[1 + \sqrt{\frac{\sin(\phi + \delta)\sin(\phi - \beta)}{\sin(\theta - \delta)\sin(\theta + \beta)}} \right]^2} \qquad [9\text{-}5]$$

where θ is the face inclination from horizontal, and β is the surcharge slope angle. The wall friction angle δ is assumed to be equal to β.

C. Develop the composite pressure diagram:

The earth pressure and surcharge pressure diagrams are combined to develop a composite earth pressure diagram which is used for design. See the standard references for procedures on locating the resultant forces.

STEP 7. Check external wall stability.

As with conventional retaining wall design, the overall stability of a geosynthetic MSE wall must be satisfactory. External stability failure modes of sliding, bearing capacity, and overturning are evaluated by assuming that the reinforced soil mass acts as a rigid body, although in reality the wall system is really quite flexible. It must resist the earth pressure imposed by the backfill which is retained by the reinforced mass and any surcharge loads. Potential external modes of failure to be considered are:

- sliding of the wall;
- bearing capacity of the wall foundation; and
- stability of the slope created by the wall (both external and compound failure planes — see Chapter 8).

These failure modes and methods of design against them are discussed by Christopher and Holtz (1985, 1989), Mitchell and Villett (1987), Christopher, et al. (1990), and Elias and Christopher (1997).

The potential for sliding along the base is checked by equating the external horizontal forces with the shear stress at the base of the wall. Sliding must be evaluated with respect to the minimum frictional resistance provided by either the reinforced soil, ϕ_r, the foundation soil, ϕ_f, or the soil-reinforcement friction angle ϕ_{sg}, as measured by interface shear tests. Often, external stability, particularly sliding, controls the length of reinforcement required. Reinforcement layers at the base of the wall may be considerably longer than required by internal earth pressure considerations alone. Generally, reinforcement layers of the same length are used throughout the entire height of the wall. The factor of safety against sliding should be at least 1.5.

Design for bearing capacity follows the same procedures as those outlined for an ordinary shallow foundation. The entire reinforced soil mass is assumed to act as a footing. Because there is a horizontal earth pressure component in addition to the vertical gravitational component, the resultant is inclined and should pass through the middle third of the foundation to insure there is no uplift (tension) in the base of this assumed rigid mass.

Appropriate bearing capacity factors or allowable bearing pressures must be used as in conventional geotechnical practice. The ultimate bearing capacity, q_{ult}, is determined using classical soil mechanics:

$$q_{ult} = c_f N_c + 0.5 \gamma_r L' N_\gamma \qquad [9\text{-}6]$$

where:

$L' = L - 2e$

$$Q_{ult} = c_f N_c (1-2e/L) + 0.5 \gamma_r N_\gamma (1-2e/L)^2 \qquad [9\text{-}7]$$

Note that ground factors have to be added to these equations for conditions where a wall is founded upon a slope. The dimensionless bearing capacity coefficients N_c and N_γ, and ground factors can be readily obtained from foundation engineering textbooks.

Due to the flexibility of MSE walls, the factor of safety for bearing capacity is lower than normally used for stiffer reinforced concrete cantilever or gravity structures. The factor of safety must be at least 2.5 with respect to the ultimate bearing capacity.

Other foundation design considerations include environmental factors such as frost action, drainage, shrinkage or swelling of the foundation soils, and potential seismic activity at the site. Each of these items must be checked to ensure adequate wall performance is maintained throughout the wall's design and service life.

Overall slope stability typically requires a factor of safety of **at least 1.3** for long-term conditions. Note that the reinforced mass should not be considered as a rigid body for overall slope stability analyses. Slope stability analysis methods that model the reinforced fill and reinforcement as discrete elements should be used, as presented in Chapter 8.

In seismically active areas, the reinforced wall and facing system, if any, must be stable during earthquakes. Seismic stability is discussed in Section 9.4-4.

STEP 8. Estimate settlement of the reinforced section.

Potential settlement of the wall structure should be assessed and conventional settlement analyses for shallow foundations carried out to ensure that immediate, consolidation, and secondary settlements of the wall are less than the performance requirements of the project, if appropriate. Both total and differential settlements along the wall length should be considered. For specific procedures, consult standard textbooks on foundation engineering.

Tolerable differential settlement for modular block walls is on the order of 1 in 200 and can be improved by placing compression materials between block units. Wrapped face walls are much more deformable and can tolerate significant differential settlement (on the order of 1 in 50, or greater).

STEP 9. Calculate the maximum horizontal stress at each level of reinforcement.

Calculate, at each reinforcement level, the horizontal stress, σ_h, along the potential active earth pressure failure surface, as shown inclined at the angle ψ in Figure 9-6. From Rankine earth pressure theory, ψ is inclined at $45° + \phi_r'/2$ for a vertical wall, where ϕ_r' is the internal friction angle appropriate for the reinforced soil section. Use the moist unit weight of the reinforced backfill plus, if present, uniform and concentrated surcharge loads.

Use K_a and the lateral earth pressure diagram, as discussed in Step 6.

STEP 10. Check internal stability and determine reinforcement requirements.

Use the lateral earth pressure diagrams developed in Step 6 for the reinforced section.

A. Determine vertical spacings, S_v, of the geosynthetic reinforcing layers and the strength of the reinforcement, T_{max}, required at each level to resist the internal lateral pressures.

The required tensile strength, T_{max}, of the geosynthetic is controlled by the vertical spacing of the layers of the reinforcing, and it is obtained from:

$$T_{max} = S_v \, \sigma_H \qquad [9\text{-}8]$$

where:

S_v = ½ (distance to reinforcing layer above + distance to reinforcing layer below)

σ_H = horizontal earth pressure at middle of the layer

Vertical spacings should be based on multiples of the compacted fill lift thickness. From Equation 9-8, it is obvious that greater vertical spacing between the horizontal layers is possible if stronger geosynthetics are used. (NOTE: Vertical spacing may be governed by the connection strength between the reinforcement and facing.) This may reduce the cost of the reinforcement, as well as increase the fill placement rate to some extent. Typical reinforcement spacing for MSE walls varies between 200 mm to 0.8 m for geogrids and rigid facings, and between 200 to 300 mm for geosynthetic wrap walls. For MBW-faced wall, the maximum vertical reinforcement spacing should be limited to twice the width (front to back) of MBW units or 0.8 m, whichever is less (AASHTO Standard Specification for Bridges, 1996, with 1997 interims).

B. Determine the length, L_e, of geosynthetic reinforcement required to develop pullout resistance beyond the Rankine failure wedge.

This design step is necessary to calculate embedment length, L_e, behind the assumed failure plane (Figure 9-6). The angle of the assumed failure plane is taken to be the Rankine failure angle, or $45° + \phi_r/2$. Also, this plane is usually assumed to initiate from the toe of the wall and proceed upward at that angle. This assumption results in conservative embedment lengths. The formula for the embedment length, L_e, is:

$$L_e \geq \frac{T_i}{2\,\gamma_r\,z\,R_c\,F^*\,\alpha}\,(FS) \qquad [9\text{-}9]$$

where:

- T_i = computed tensile load in the geosynthetic;
- γ_r = unit weight of backfill (reinforced section);
- z = depth of the layer being designed;
- R_c = reinforcement coverage ratio;
- F^* = coefficient of pullout interaction between soil and geosynthetic;
- α = scale correction factor;
- FS = factor of safety against pullout failure.

The factor of safety for embedment should be 1.5, with a minimum embedment length of approximately 1 m. For standard backfill materials, with the exception of uniform sands (i.e., coefficient of uniformity, $C_u < 4$), it is acceptable to use conservative default values for F* and α as shown in Table 9-2.

TABLE 9-2
DEFAULT VALUES FOR F* AND α PULLOUT FACTORS

Reinforcement Type	Default F*	Default α
Geogrid	0.8 tan ϕ	0.8
Geotextile	0.67 tan ϕ	0.6

For wrap-around walls, the overlap length, L_o, must be long enough to transfer stresses from the lower portion to the longer layer above it. The equation for geosynthetic overlap length, L_o, is:

$$L_o = \frac{T_i}{2 \, \gamma_r \, z \, R_c \, F^* \, \alpha} (FS) \qquad [9\text{-}10]$$

Again, a minimum value of approximately 1 m is recommended for L_o to insure adequate anchorage of reinforcement layers.

STEP 11. Prepare plans and specifications.

Specifications are discussed in Section 9.9.

9.4-4 Seismic Design (Allen and Holtz, 1991)
In seismically active areas, an analysis of the geosynthetic MSE wall stability under seismic conditions must be performed. For temporary structures, a formal analysis is probably not necessary. For permanent structures, seismic analyses can range from a simple pseudo-static analysis to a complete dynamic soil-structure interaction analysis such as might be performed on earth dams and other critical structures.

Seismic analysis procedures for MSE walls with metallic reinforcement and concrete facings are well established; see Vrymoed (1990) for a review of these procedures. The generally conservative pseudo-static Mononabe-Okabe analysis is recommended for geosynthetic MSE walls in the AASHTO and FHWA guidelines. This analysis correctly includes the horizontal inertial forces for internal seismic resistance, as well as the pseudo-static thrust imposed by the retained fill on the reinforced section.

Because of their inherent flexibility, properly designed and constructed geosynthetic walls are probably better able to resist seismic loadings, but high walls in earthquake-prone regions should be checked. The facing connections must also resist the inertial force of the wall fascia which can occur during the design seismic event. Stress build-up behind the face, resulting from strain incompatibility between a relatively stiff facing system and the extensible geosynthetic reinforcement must also be resisted by facing connections. Additional research is needed to evaluate the effect of seismic forces on geosynthetic walls with stiff facings.

9.5 LATERAL DISPLACEMENT

Lateral displacement of the wall face occurs primarily during construction, although some also can occur due to post construction surcharge loads. Post-construction deformations can also occur due to structure settlement. As noted by Christopher, et al. (1990), there is no standard method for evaluating the overall lateral displacement of reinforced soil walls.

The major factors influencing lateral displacements during construction include compaction intensity, reinforcement to soil stiffness ratio (*i.e.*, the modulus and the area of reinforcement as compared to the modulus and area of the soil), reinforcement length, slack in reinforcement connections at the wall face, and deformability of the facing system. An empirical relationship for estimating relative lateral displacements during construction of walls with granular backfills is presented in the Construction Guidelines for Reinforced Soil Structures (Christopher, et al., 1989). The relationship was developed from finite element analyses, small-scale model tests, and very limited field evidence from 6 m high test walls. Note that as L/H decreases, the lateral deformation increases. The procedure predicted wall face movements of a 12.6 m high geotextile wall that were slightly greater than these observed (Holtz, et al., 1991).

Two major factors influencing lateral displacements -- compaction intensity and slack in the reinforcement at the wall face -- are contractor controlled. Therefore, geosynthetic MSE wall specifications should state acceptable horizontal and vertical erected face tolerances.

9.6 MATERIAL PROPERTIES

9.6-1 Reinforced Wall Fill Soil

Gradation: All soil fill material used in the structure volume shall be reasonably free from organic or other deleterious materials and shall conform to the limits (AASHTO, 1990) presented in Table 9-3.

TABLE 9-3
MSE SOIL FILL REQUIREMENTS

Sieve Size	Percent Passing
19 mm[1]	100
4.75 mm	100 - 20
0.425 mm	0 - 60
0.075 mm	0 - 15

Plasticity Index (PI) ≤ 6 (AASHTO T-90)

Soundness: magnesium sulfate soundness loss < 30% after 4 cycles

NOTE:
1. The maximum size can be increased up to 100 mm, provided tests have been or will be performed to evaluate geosynthetic strength reduction due to installation damage (see Appendix K).

Chemical Composition (Elias and Christopher, 1997): The chemical composition of the fill and retained soils should be assessed for effect on durability of reinforcement (pH, oxidation agents, etc.). Some soil environments posing potential concern when using geosynthetics are listed in Appendix K. It is recommended that application of polyester based geosynthetics be limited to soils with a pH range between 3 and 9. Polyolefin based geosynthetics (i.e., polyethylene and polypropylene) should be limited to use with soils of pH >3.

Compaction (Elias and Christopher, 1997): A minimum density of 95 percent of AASHTO T-99 maximum value is recommended for retaining walls, and 100 percent of T-99 is recommended for abutments and walls supporting structural foundations. Soil fill shall be placed and compacted at or within + or - 2 percentage points of optimum moisture content, w_{opt}, according to AASHTO T-99. If the reinforced fill is free draining with less than 5 percent passing a 0.075 mm sieve, water content of the fill may be within + or - 3 percentage points of the optimum. Compacted lift height of 200 to 300 mm is recommended for granular soils.

A small single or double drum walk-behind vibratory roller or vibratory plate compactor should be used within 1 m of the wall face. Within this 1 m zone, quality control should be maintained by a methods specification, such as three passes of light drum compactor. Excessive compactive effort or use of too heavy of equipment near the face could result in excessive face movement, and overstressing of reinforcement layers.

Compaction control testing of the reinforced backfill should be performed on a regular basis during the entire construction project. A minimum frequency of one test with the reinforced zone per every 1.5 m of wall height for every 30 m of wall is recommended. Inconsistent compaction and undercompaction caused by insufficient compactive effort will lead to gross misalignments and settlement problems, and should not be permitted.

Shear Strength: Peak shear strength parameters should be used in the analysis (Christopher, et al., 1989). Parameters should be determined using direct shear and triaxial tests.

Shear strength testing is recommended. However, use of assumed shear values based on Agency guidelines and experience may be acceptable for some projects. Verification of site soil type(s) should be completed following excavation or identification of borrow pit, as applicable.

Unit Weights: Dry unit weight for compaction control, moist unit weight for analyses, and saturated unit weight for analyses (where applicable) should be determined for the fill soil. The unit weight value of should be consistent with the design angle of internal friction, ϕ.

9.6-2 Geosynthetic Reinforcement

Geosynthetic reinforcement systems consist of geogrid or geotextile materials arranged in horizontal planes in the backfill to resist outward movement of the reinforced soil mass. Geogrids transfer stress to the soil through passive soil resistance on grid transverse members and through friction between the soil and the geogrid's horizontal surfaces (Mitchell and Villet, 1987). Geotextiles transfer stress to the soil through friction. Geosynthetic design strength must be determined by testing and analysis methods that account for the long-term geosynthetic-soil stress transfer and durability of the full geosynthetic structure. Long-term soil stress transfer is characterized by the geosynthetic's ability to sustain long-term load in-service without excessive creep strains. Durability factors include site damage, chemical degradation, and biological degradation. These factors may cause deterioration of either the geosynthetic's tensile elements or the geosynthetic structure's geosynthetic/soil stress transfer mechanism.

An inherent advantage of geosynthetics is their longevity in fairly aggressive soil conditions. The anticipated half-life of some geosynthetics in normal soil environments is in excess of 1000 years. However, as with steel reinforcements, strength characteristics must be adjusted to account for potential degradation in the specific environmental conditions, even in relatively neutral soils. Questionable soil environments are listed in Appendix K.

Allowable Tensile Strength: Allowable tensile strength (T_a) of the geosynthetic shall be determined using a partial factor of safety approach (Bonaparte and Berg, 1987). Reduction factors are used to account for installation damage, chemical and biological conditions and to control potential creep deformation of the polymer. Where applicable, a reduction is also applied for seams and connections. The total reduction factor is based upon the mathematical product of these factors. The long-term tensile strength, T_{al}, thus can be obtained from:

$$T_{al} = \frac{T_{ult}}{RF} \qquad [9\text{-}11]$$

with RF equal to the product of all applicable reduction factors:

$$RF = RF_{CR} \times RF_{ID} \times RF_{D} \qquad [9\text{-}12]$$

where:

T_{al} = long-term tensile strength,(kN/m);
T_{ult} = ultimate geosynthetic tensile strength, based upon MARV, (kN/m);
RF_{CR} = creep reduction factor, ratio of T_{ult} to creep-limiting strength, (dimensionless);

RF_{ID} = installation damage reduction factor, (dimensionless); and

RF_D = durability reduction factor for chemical and biological degradation, (dimensionless).

RF values for durable geosynthetics in non-aggressive, granular soil environments range from 3 to 7. Appendix K suggests that a default value RF = 7 may be used for routine, non-critical structures **which meet the soil, geosynthetic and structural limitations listed in the appendix.** However, as indicated by the range of RF values, there is a potential to significantly reducing the reinforcing requirements and the corresponding cost of the structure by obtaining a reduced RF from test data.

The procedure presented above and detailed in Appendix K is derived from Elias and Christopher (1997), Berg (1993), the Task Force 27 (1990) guidelines for geosynthetic reinforced soil retaining walls, the Geosynthetic Research Institute's Methods GG4a and GG4b - *Standard Practice for Determination of the Long Term Design Strength of Geogrids* (1990, 1991), and the Geosynthetic Research Institute's Method GT7 - *Standard Practice for Determination of the Long Term Design Strength of Geotextiles* (1992).

Additionally, the following factors should be considered. The *long-term* strength determined by dividing the ultimate strength by RF does not include an overall factor of safety to account for variation from design assumptions (*e.g.*, heavier loads than assumed, construction placement, fill consistency, etc.). A safety factor is applied to the reinforcement when designing MSE structures to quantify a safe *allowable* strength. Thus the allowable strength of a geosynthetic for MSE applications can be defined as:

$$T_a = \frac{T_{al}}{FS} \qquad [9\text{-}13]$$

where:

T_a = allowable geosynthetic tensile strength,(kN/m); and

FS = overall factor of safety to account for uncertainties in the geometry of the structure, fill properties, reinforcement properties, and externally applied loads.

For permanent, MSE wall structures, a minimum factor of safety, FS, of 1.5 is recommended. Of course, the FS value should be dependent upon the specifics of each project.

Connection Strength: The design (or factored allowable) strength may be limited by the strength of the connection between the reinforcement and wall facing unit. The latest AASHTO (1996, with 1997 interims) and FHWA (Elias and Christopher, 1997) guidelines contain new procedures for quantifying the allowable connection strength. Some of the parameters required for the AASHTO/FHWA procedure are not readily definable with current laboratory test procedures. These new procedures are very restrictive for some combinations of reinforcement and face units, as compared to past criteria. Therefore, these guidelines are currently under review, and additional work is forthcoming.

The AASHTO/FHWA procedure for developing an allowable connection strength is to use the lessor of:

i) The long-term tensile strength, T_{al}, of the reinforcement (Eq. 9-11).

ii) The reduced ultimate connection strength based upon connection/seam strength CR_u as determined from ASTM D 4884 for seams and MBW connection strength test (Elias and Christopher, 1997; Simac et al., 1993) for partial or full friction connections, with

$$CR_u = \frac{T_{ultc}}{T_{lot}} \qquad [9\text{-}14]$$

where:

T_{ultc} = load per unit reinforcement width which results in rupture of the reinforcement in this test at a specified confining pressure (note that at low confining pressure, T_{ultc} may not occur);

T_{lot} = the ultimate wide width tensile strength (ASTM D 4595) for the reinforcement material lot used for the connection strength testing; and

CR_u = reduction factor to account for reduced ultimate strength resulting from the connection.

iii) The connection strength based on pullout as developed by testing CR_s (Elias and Christopher, 1997), with

$$CR_s = \frac{T_{sc}}{T_{lot}} \qquad [9\text{-}15]$$

where:

T_{sc} = load per unit reinforcement width at a specified maximum deformation or at the peak pullout load, whichever occurs first (note that a maximum deformation of 20 mm is recommended);

T_{lot} = the ultimate wide width tensile strength (ASTM D 4595) for the reinforcement material lot used for the connection strength testing; and

CR_s = reduction factor to account for reduced strength resulting from the connection.

iv) The maximum connection strength as developed by testing reduced for long-term environmental aging, creep and divided by a factor of safety of at least 1.5 for permanent structures, as follows.

For reinforcement rupture (during connection testing):

$$T_{ac} \leq \frac{T_{ult} \; x \; CR_u}{RF_D \; x \; RF_{CR} \; x \; 1.5} \qquad [9\text{-}16a]$$

For reinforcement pullout (during connection testing):

$$T_{ac} \leq \frac{T_{ult} \; x \; CR_s}{1.5} \qquad [9\text{-}16b]$$

Note that the environment at the connection may not be the same as the environment within the MSE mass. Therefore, the long-term environmental aging factor, RF_D, may be significantly different than that used in computing the allowable reinforcement strength, T_a.

The connection strength as developed above is a function of normal pressure which is developed by the weight of the units. Thus, it will vary from a minimum in the upper portion of the structure to a maximum near the bottom of the structure for walls with not batter. Further, since many MBW walls are constructed with a front batter, the column weight above the base of the wall or above any other interface may not correspond to the weight of the facing units above the reference elevation. The concept is known as the *hinge height* (Simac et al., 1993). Hence, for walls with a batter, the normal stress is limited to the lesser of the hinge height, H_h, or the height of the wall above the interface. This vertical pressure range should be used in developing CR_u and CR_s.

Soil-Reinforcement Interaction: Two types of soil-reinforcement interaction coefficients or interface shear strengths must be determined for design: pullout coefficient, and interface friction coefficient (Task Force 27 Report, 1990). Pullout coefficients are used in stability analyses to compute mobilized tensile force **at the front** and **tail** of each reinforcement layer. Interface friction coefficients are used to check factors of safety against outward sliding of the entire reinforced mass.

Detailed procedures for quantifying interface friction and pullout interaction properties are presented in Appendix K. The ultimate pullout resistance, P_r, of the reinforcement per unit width of reinforcement is given by:

$$P_r = 2 \bullet F^* \bullet \alpha \bullet \sigma'_v \bullet R_c \bullet L_e$$

where:

$L_e \bullet 2$	=	the total surface area per unit width of the reinforcement in the resistance zone behind the failure surface
L_e	=	the embedment or adherence length in the resisting zone behind the failure surface
F^*	=	the pullout resistance (or friction-bearing-interaction) factor
α	=	a scale effect correction factor
σ'_v	=	the effective vertical stress at the soil-reinforcement interfaces
R_c	=	reinforcement coverage ratio

Default values for F* and α are presented in Table 9-2.

9.7 COST CONSIDERATIONS

At the FHWA-Colorado Department of Highways Glenwood Canyon geotextile test walls (Bell, et al., 1983), the cost of the geotextile was only about 25% of the wall's total cost. Therefore, some conservatism on geosynthetic strength or on vertical spacing is not necessarily excessively expensive. A major part of the Glenwood Canyon costs involved the hauling and placement of backfill, as well as shotcrete facing. In some situations, especially where contractors are unfamiliar with geosynthetic reinforcement, artificially high unit costs have been placed on bid items such as the shotcrete facing, which effectively has made the reinforced soil wall uneconomical. The geosynthetic soil reinforcement is approximately 15% of the total in-place cost of highway MSE walls with MBW unit facing and select granular soil fill. A cost comparison for reinforced versus other types of retaining walls is presented in Figure 9-7. For low walls, geosynthetics are usually less expensive than conventional walls and metallic MSE wall systems. At the time of its construction, the Rainier Avenue wall was the highest geotextile wall ever constructed (Allen, et al., 1992). It was unusually economical, partially because, as a temporary structure, no special facing was used. Permanent facing on a wall of that height would have increased its cost by \$50/m^2 or more.

Other factors impacting cost comparison include site preparation; facing cost, especially if precast panels or other special treatments are required; special drainage required behind the backfill; instrumentation; etc.

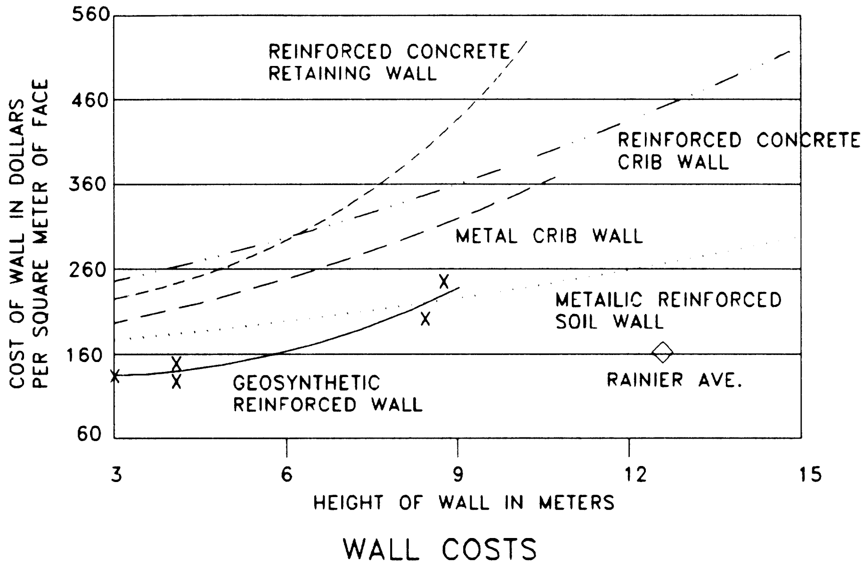


Figure 9-7 Cost comparison of reinforced systems.

9.8 COST ESTIMATE EXAMPLES

9.8-1 Geogrid, MBW unit-Faced Wall

A preliminary cost estimate for an MSE wall is needed to assess its viability on a particular project. Therefore, a rough design is required to estimate fill and soil reinforcement quantities. The project's scope is not fully defined, and several assumptions will be required.

STEP 1. Wall description

The wall will be approximately 200 m long, and varies in exposed wall height from 4 to 6 meters. A gradual slope of 5H:1V will be above the wall. The wall will have a nominal (*e.g.*, < 3°) batter. Seismic loading can be ignored.

An MBW unit facing will be specified. The geosynthetic will be a geogrid. Reinforced wall fill will be imported. Wall fill soils are not aggressive and pose no specific durability concerns.

STEP 2. Foundation soil

Wall will be founded on a competent foundation, well above the estimated water table. The in situ soils are silty sands, and an effective friction angle of 30° can be assumed for conceptual design. A series of soil borings along the proposed wall alignment will be completed prior to final design.

STEP 3. Reinforced fill and retained backfill properties

A well-graded gravely sand, with 20 mm maximum size, will be specified as wall fill, as it is locally available at a cost of approximately \$3 per 1,000 kg delivered to site. An effective angle of friction of 34° and a unit weight of 20 kN/m^3 can be assumed. The fill is nonaggressive, and a minimum durability partial safety factor can be used.

The retained backfill will be on-site silty sand embankment material. An effective angle of friction of 30° and a unit weight of 19 kN/m^3 can be assumed.

STEP 4. Establish design factors of safety.

For external stability, use minimums of:

sliding	1.5
bearing capacity	2.5
overall stability	1.3

For internal stability, use minimums of:

FS = 1.5 against reinforcement failure

FS = 1.5 against pullout failure

STEP 5. Determine preliminary wall dimensions.

Average exposed wall height is approximately 5 m. An embedment depth of 0.5 (the minimum recommended) should be added to the exposed height. Total design height is 5.5 m.

Assume L/H ratio of approximately 0.7. Use an $L = 0.7\ (5.5) \approx 4$ m.

STEP 6. Develop earth pressure diagrams.

$$\sigma_H = K_a\, \sigma_v$$

At the base of the wall,

$$\sigma_v = \gamma_r\, H + 0.5\, L\, (\tan \beta)\, \gamma_b = 20\ kN/m^3\ (5.5\ m) + 0.5\ (4\ m)(0.2)\ 20\ kN/m^3 = 118\ kN/m^2$$

For internal stability $K_a = \tan^2(45 - \phi/2) = 0.28$

For external stability $K_a = \tan^2(45 - \phi/2) = 0.33$

STEP 7. Check external stability.

By observation and experience, it is assumed that the L/H ratio of 0.7 will provide adequate external safety factors for the project conditions.

STEP 8. Estimate settlement.

Again, by observation and experience, settlement is not a problem for these project conditions.

STEP 9. Calculate horizontal stress at each layer of reinforcement.

Not required for conceptual design; see next step.

STEP 10. Check internal stability and determine reinforcement requirements.

The maximum lateral stress, σ_H, to be resisted by the geogrid is at the bottom of the wall and is equal to:

$$\sigma_H = 0.28\ (118\ kN/m^2) = 33\ kN/m^2$$

Assume 100% geogrid coverage in plan view. Assume a geogrid spacing and calculate T_{max} and T_a per Step 10 (i.e., use a maximum spacing of 0.6 m to match block height). Assume vertical spacing of 0.6 m, which is about one geogrid every three blocks. Therefore, 9 layers of geogrid will be used. a geogrid with a long-term allowable strength of 20 kN/m will be used. The required strength of the lowest geogrid is therefore equal to:

$$T_{max} = 0.6\ m\ (\ 33\ kN/m^2) = 19.8\ kN/m \geq T_a$$

Use $T_a = 20\ kN/m \rightarrow T_{al}\ /\ 1.5 = 30\ kN/m$ for the bottom layers of geogrid. The top layers could be reduced by about ½. Therefore, use 5 geogrid layers at the bottom at 30 kN/m and 4 geogrid layers at the top at 20 kN/m.

COST ESTIMATE:

Material Costs:

Leveling Pad - 200 m ($10 / m) = $2,000

Reinforced wall fill -

200 m (4 m) (5.45 m) (20 kN/m^3) = 87,200 kN → 87,200,000 N ÷ 9.8 = 8,900,000 kg

8,900,000 kg ($3 / 1,000 kg) = $ 27,000

Geogrid soil reinforcement -

5 layers (4 m) (200 m) = 4,000 m^2 of 30 kN/m; and

4 layers (4 m) (200 m) = 3,200 m^2 of 20 kN/m

From the range presented in Appendix K, assume material costs, delivered to site, of $ 3.35 / m^2 and $ 2.25 / m^2. Therefore, cost is 4,000 m^2 ($3.35 / m^2) + 3,200 m^2 ($2.25 / m^2) = $20,600

MBW face units -

From local market, MBW units range in cost from $50 to $70 / m^2

Assume a cost of $60 / m^2

200 m (5.45 m) ($60 / m^2) = $65,000

Gravel drain fill within or behind the MBW units -
Assume 0.3 m thickness required. Assume a cost of \$7.50 per compacted m^3.
200 m (5.45 m) (0.3 m) (\$7.50 / m^3) = \$3,000

Engineering and Testing Costs:
A line-and-grade specification will be used. Based upon previous projects, assume cost of design engineering, soil testing, and site assistance will be approximately \$10 per m^2.
200 m (5.45 m) (\$10 / m^2) = \$11,000

Installation Costs:
Based upon previous bids, assume cost to install will be approximately \$50 per m^2
200 m (5.45 m) (\$50 / m^2) = \$55,000

TOTAL ESTIMATED COST:

Materials + Engineering/Site Assistance + Installation =
(\$2,000 + \$27,000 + \$20,600 + \$65,000 + \$3,000) + \$11,000 + \$55,000 = \$183,600

{Check: This is equal to an installed cost of \$167 / m^2, which is reasonable based upon past experience.} Based upon this cost estimate, the geosynthetic MSE wall option is the most economical for this project. Therefore, it is recommended that final design proceed using a geosynthetic MSE wall. Note that estimate does not include site preparation, placement of random backfill, or final completion items (*e.g.*, seeding, railings).

9.8-2 Geotextile Wrap Wall

A preliminary cost estimate for a temporary MSE wall is needed to assess its viability on a particular project. Therefore, a rough design is required to estimate fill and soil reinforcement quantities. The project scope is not fully defined, and several assumptions will be required.

STEP 1. Wall description.

The temporary wall will be approximately 50 m long and approximately 8 m high. A flat slope and no traffic will be above the wall. Seismic loading can be ignored.

A wrap-around facing will be used and an ultraviolet-stabilized geotextile specified. Thus, a gunite or other type of protective facing for this temporary structure will not be required. Wall fill soils are not aggressive and pose no specific durability concerns.

STEP 2. Foundation soil.

Wall will be founded on a competent foundation that overlies a soft compressible layer of soil. Details of foundation bearing capacity and global stability do not have to be addressed for this conceptual cost estimate, but will be addressed in final design. The in situ soils are silts, and an effective friction angle of 28° can be assumed for conceptual design.

STEP 3. Reinforced fill and retained backfill properties.

A well-graded sand will be specified as wall fill, as it is locally available at a cost of approximately $2 per 1,000 kg delivered to site. An effective angle of friction of 32° and a unit weight of 19.5 kN/m^3 can be assumed. The fill is nonaggressive, and a minimum durability partial safety factor can be used.

The retained backfill will be on site silt embankment material. An effective angle of friction of 28° and a unit weight of 18.5 kN/m^3 can be assumed.

STEP 4. Establish design factors of safety.

For external stability, use minimums of:

sliding	1.5
bearing capacity	2.5
overall stability	1.3

For internal stability, use minimums of:

FS = 1.5 against reinforcement failure
FS = 1.5 against pullout failure

STEP 5. Determine preliminary wall dimensions.

Average exposed wall height is approximately 8 m. An embedment distance of 0.45 (the minimum recommended) should be added to the exposed height. Design height is 8.45 m.

Assume L/H ratio of approximately 0.7. Use an $L = 0.7\ (8.45) \approx 6$ m.

STEP 6. Develop earth pressure diagrams.

σ_v at base of wall $= \gamma_r H = 19.5\ \text{kN/m}^3\ (8\ \text{m}) = 156\ \text{kN/m}^2$

For internal stability $K_a = \tan^2(45 - \phi/2) = 0.31$

For external stability $K_a = \tan^2(45 - \phi/2) = 0.36$

STEP 7. Check external stability.

By observation and experience, it is assumed that the L/H ratio of 0.7 will provide adequate external safety factors for the project conditions. {This assumption will checked in final design.} Therefore,

$L = 0.7\ (8\ \text{m}) = 5.6\ \text{m}$

STEP 8. Estimate settlement.

Again, by observation and experience, settlement is likely not a problem for these project conditions. Settlement will be quantified during final design.

STEP 9. Calculate horizontal stress at each layer of reinforcement.

Not required for conceptual design; see next step.

STEP 10. Check internal stability and determine reinforcement requirements.

Lateral load to be resisted by the geotextile is equal to:

$$\tfrac{1}{2}\, K_a\, \gamma\, H^2 \;=\; \tfrac{1}{2}\,(0.31)\,(19.5\ \text{kN/m}^3)\,(8.45\ \text{m})^2 \;=\; 216\ \text{kN/m}$$

Assuming 100% geotextile coverage in plan view, the geotextiles must safely carry 216 kN/m per unit width of wall. Assume a geotextile with a long-term allowable strength of 20 kN/m will be used. The safe design strength of the geotextile is therefore equal to:

$$T_{max} \;=\; T_{al}\,/\,FS \;=\; 20\,/\,1.5 \;=\; 13.3\ \text{kN/m}$$

The approximate number of geotextile layers required is equal to:

$$216 \;\div\; 13.3 \;=\; 16.2$$

Round this number up and add an additional layer for conceptual design to account for practical layout considerations with final design. Assume a vertical spacing of 0.5 m will be used. Therefore, 16 layers of geotextile will be used, with $T_{max} \approx 13.5$ kN/m, or greater.

COST ESTIMATE:

Material Costs:

Reinforced wall fill -

50 m (8.45 m) (6 m) (20 kN/m^3) = 50,700 kN $\rightarrow$ 50,700,000 N $\div$ 9.8 = 5,173,500 kg

5,173,500 kg (\$2 / 1,000 kg) = \$ 10,400

Geotextile soil reinforcement (include face area and wrap-tail length) -

16 layers (6 + 0.5 + 1.5 m) (50 m) = 6,400 m^2

From the range presented in Appendix K, assume a material cost, delivered to site, of \$2 / m^2

6,800 m^2 (\$2 / m^2) = \$13,600

Engineering and Testing Costs:

A line-and-grade specification will be used. Based upon previous projects, assume cost of design engineering, soil testing, and site assistance will be approximately \$30 per m^2, because of the height and relatively low total area of wall that will be constructed.

50 m (8.45 m) (\$30 / m^2) = \$12,700

Installation Costs:

Based upon previous bids, assume cost to install will be approximately \$60 per m^2

50 m (8.45 m) (\$60 / m^2) = \$25,400

TOTAL ESTIMATED COST:

Materials + Engineering/Site Assistance + Installation =
\$24,000 + \$12,700 + \$25,400 = \$62,100

{Check: This is equal to an installed cost of \$147 / m^2, which is reasonable based the small size of this project and upon past experience.} Based upon this cost estimate, the geosynthetic MSE wall option is the most economical for this project. Therefore, it is recommended that final design proceed using a geosynthetic MSE wall.

9.9 SPECIFICATIONS

9.9-1 Geosynthetic, MBW Unit-Faced Wall

The following example was obtained from New York DOT. It is a special provision, from a specific project, for materials and construction of a geogrid-MBW unit reinforced soil wall.

ITEM 17554.02 MECHANICALLY STABILIZED SEGMENTAL BLOCK RETAINING WALL SYSTEM (EXTENSIBLE REINFORCEMENT)

DESCRIPTION

Construct a Mechanically Stabilized Segmental Block Retaining Wall System (Extensible Reinforcement), (MSSBRWS) where indicated on the plans.

A MSSBRWS consists of an un-reinforced concrete or compacted granular leveling pad, facing and cap units, backfill, underdrains, geotextiles, and an extensible reinforcement used to improve the mechanical properties of the backfill.

Other definitions that apply within this specification are:

A. Leveling Pad — An un-reinforced concrete or compacted granular fill footing or pad which serves as a flat surface for placing the initial course of facing units.

B. Facing Unit — A segmental precast concrete block unit that incorporates an alignment and connection device and also forms part of the MSSBRWS face area. A corner unit is a facing unit having two faces.

C. Alignment and Connection Device — Any device that is either built into or specially manufactured for the facing units, such as shear keys, leading/trailing lips, or pins. The device is used to provide alignment and maintain positive location for a facing unit and also provide a means for connecting the extensible reinforcement.

D. Extensible Reinforcement — High density polyethylene, polypropylene or high tenacity polyester geogrid mats formed by a regular network of integrally connected polymer tensile elements with apertures of sufficiently large size to allow for mechanical interlock with the backfill, or geotextiles of specified lengths which connect to the facing unit.

E. Unit Fill — Well-graded aggregate fill placed within and/or contiguous to the back of the facing unit.

F. Cap Unit — A segmental precast concrete unit placed on and attached to the top of the finished MSSBRWS.

G. Backfill — Material placed and compacted in conjunction with extensible reinforcement and facing units.

H. Underdrain — A system for removing water from behind the MSSBRWS.

I. Geotextile — A permeable textile material used to separate dissimilar granular materials.

MATERIALS

Not all materials listed are necessarily required for each MSSBRWS. Ensure that the proper materials are supplied for the chosen system design.

A. Leveling Pad

1. For MSSBRWS greater than or equal to 4.6 m in total height, supply a leveling pad conforming to the following:

   a. Un-reinforced Concrete

      Supply concrete conforming to Section 501 (Class A Concrete).

2. For MSSBRWS less than 15 feet in total height, supply a leveling pad conforming to one of the following:

   a. Un-reinforced Concrete

      Supply concrete conforming to Section 501 (Class A Concrete).

   b. Granular Fill

      Supply select granular fill conforming to Subsection 203-2.02C (Select Granular Fill and Select Structure Fill).

c. Crushed Stone

Supply crushed stone conforming to Section 623 (Screened Gravel, Crushed Gravel, Crushed Stone, Crushed Slag), Item 623.12, Crushed Stone (In-Place measure), Size Designation 1.

B. Facing and Cap Units

Supply units fabricated and conforming to Subsection 704-07 (Precast Concrete Retaining Wall Blocks). Notify the Director, Materials Bureau, of the name and address of the units' fabricator no later than 14 days after contract award.

C. Alignment and Connection Devices

Supply devices conforming to the designer-supplier's Installation Manual.

D. Extensible Reinforcement

Supply reinforcement manufactured and conforming to Sub-section 725-03 (Geosynthetic Reinforcement).

E. Unit Fill

Supply fill conforming to material and gradation requirements for Type CA-2 Coarse Aggregate under Sub-section 501-2.02, B.2 (Coarse Aggregate).

F. Cast-in-place Concrete

Supply concrete conforming to Section 501 (Class A Concrete).

G. Backfill

Supply backfill material as shown on the plans and conforming to Subsection 203-1.08 (Suitable Material). Backfill material must come form a single source, unless prior written approval for use of multiple sources is obtained from the Director, Geotechnical Engineering Bureau.

Stockpile backfill material conforming to the current Soil Control Procedure (SCP) titled "Procedure for the Control of Granular Materials."

1. Material Test Procedures

The State will perform procedures conforming to the appropriate Departmental publications in effect on the date of advertisement of bids. These publications are available upon request to the Regional Director, or the Director, Geotechnical Engineering Bureau.

2. Material Properties

a. Gradation

Stockpiled backfill material must meet the following gradation requirements:

TABLE 17554-2

Sieve Size Designation	Percent Passing by Weight
64 mm	100
6.5 mm	30 - 100
0.425 mm	0 - 60
0.075 mm	0 - 15

b. Plasticity Index.

The Plasticity Index must not exceed 5.

c. Durability.

The Magnesium Sulfate Soundness loss must not exceed 30 percent.

H. Geotextile

Supply geotextile material conforming to Section 207 (Geotextile), Item 207.03, Geotextile Underdrain.

I. Drainage

Supply underdrain and geotextile material as shown on the plans:

1. Underdrain Pipe

Supply optional underdrain pipe conforming to Section 605 (Underdrains).

2. Geotextile Underdrain

Supply geotextile underdrain conforming to Section 207 (Geotextile), Item 207.03, Geotextile Underdrain.

J. Identification Markers

Supply identification markers conforming to the designer-supplier's Installation Manual.

K. Basis of Acceptance

Accept cast-in-place concrete conforming to Section 501 (Portland Cement Concrete), Class A.

Accept other materials by manufacturer's certification. The State reserves the right to sample, test, and reject certified material conforming to the Departmental written instructions.

Only approved MSSBRWS designer-suppliers appearing on the attached Approved List of Products will be acceptable for use under this item.

Obtain all necessary materials (except backfill, underdrains, geotextiles, and cast-in-place concrete) form the approved designer-supplier. Upon award of the contract, notify the Deputy Chief Engineer, Technical Services (DCETS) of the name and address of the chosen designer-supplier. Once designated the designer-supplier shall not be changed.

Obtain from the designer-supplier and submit to the DCETS for approval, the MSSBRWS design and installation procedure. All designs must be stamped by a Professional Engineer licensed to practice in New York State. The DCETS requires 20 working days to approve the submission after receipt of all pertinent information. Begin work only after receiving DCETS approval.

Submit shop drawings and proposed methods for construction to the Engineer for written approval at least 30 working days before starting work. Shop drawings must conform to the size and type requirements given in Subsection 718-01 (Prestressed Concrete Units (Structural)) under Drawing Types, Subparagraph 2.A (Working Drawings, Size and Type).

Supply on-site technical assistance from a representative of the designated designer-supplier during the beginning of installation until such time as the Engineer determines that outside consultation is no longer required.

Provide the Engineer with two copies of the designated designer-supplier Installation Manual two weeks before beginning construction.

A. Excavation, Disposal and MSSBRWS Area Preparation

Excavate, dispose and prepare the area on which the MSSBRWS will rest conforming to Section 203 (Excavation and Embankment), except as modified here:

1. Grade and level, for a width equaling or exceeding the reinforcement length, the area on which the MSSBRWS will rest. Thoroughly compact this area to the Engineer's satisfaction. Treat all soils found unsuitable, or incapable of being satisfactorily compacted because of moisture content, in a manner directed by the Engineer, in conjunction with recommendations of the Regional Soils Engineer.

2. Remove rock to the limits shown on the plans.

3. Excavate the area for the leveling pad conforming to Section 206, (Trench, Culvert and Structure Excavation).

B. Facing and Cap Unit Storage and Inspection

Handle and store facing and cap units with extreme care to prevent damage. The State will inspect facing and cap units on their arrival at the work site and prior to their installation to determine any damage that may have occurred during shipment. Facing and cap units will be considered damaged if they contain any cracks or spalls and/or heavy combed areas with any dimension greater than 25 mm. The State will reject any damaged facing and cap units. Replace rejected units with facing and cap units acceptable to the Engineer.

C. Facing Unit Erection

1. Provide an un-reinforced concrete or compacted granular fill leveling pad as shown on the plans.

   a. Place concrete in conformance with Subsection 555-3, (Construction Details). The Engineer may waive any part of Subsection 555-3 that he determines is impractical.

   b. Place compacted granular fill in conformance with Subsection 203-3.12 (Compaction).

2. Install by placing, positioning, and aligning facing units in conformance with the designer-supplier's Installation Manual, unless otherwise modified by the contract documents or the Engineer, and check that requirements of Table 17554-4 are not exceeded. After placement, maintain each facing unit in position by a method acceptable to the Engineer.

3. Correct all misalignments of installed facing units that exceed the tolerances allowed in Table 17554-4 in a manner satisfying the Engineer.

TABLE 17554-4

Vertical control	6 mm over a distance of 3 m
Horizontal location control	13 mm over a distance of 3 m
Rotation from established plan wall batter	13 mm over 3 m in height

4. Control all operations and procedures to prevent misalignment of the facing units. Precautionary measures include (but are not limited to) keeping vehicular equipment at least 1 m behind the back of the facing units. Compaction equipment used within 1 m of the back of the facing units must conform to Subsection 203-3.12B.6 (Compaction Equipment for Confined Areas).

D. Unit Fill

1. Place unit fill to the limits indicated on the plans. Before installing the next course of facing units, compact the unit fill in a manner satisfying the Engineer and brush clean the tops of the facing units to ensure an even placement area.

2. Protect unit fill from contamination during construction.

E. Extensible Reinforcement

1. Before placing extensible reinforcement, backfill placed and compacted within a 1 m horizontal distance of the back of facing units must be no more than 25 mm above the required extensible reinforcement elevation. Backfill placed and compacted beyond the 1 m horizontal distance may be roughly graded to the extensible reinforcement elevation.

2. Place extensible reinforcement normal to facing units unless indicated otherwise on the plans. The Engineer will reject broken or distorted extensible reinforcement. Replace all broken or distorted extensible reinforcement.

3. Install extensible reinforcement within facing units conforming to the designer-supplier's Installation Manual. Pull taut and secure the extensible reinforcement before placing the backfill in a manner satisfying the Engineer.

F. Backfill

1. Place backfill materials (other than rock) at a moisture content less than or equal to the Optimum Moisture Content. Remove backfill materials placed at a moisture content exceeding the Optimum Moisture Content and either rework or replace, as determined by the Engineer. Determine Optimum Moisture Content in conformance with Soil Test Methods for compaction that incorporate moisture content determination. Use Soil Test Methods in effect on the date of advertisement of bids. Cost to rework or replace backfill materials shall be borne by the Contractor.

2. Place granular backfill material in uniform layers so that the compacted thickness of each layer does not exceed 0.25 m or one block height, whichever is less. Compact each layer in conformance with Subsection 203-3.12 (Compaction). The Engineer will determine by visual inspection that proper compaction has been attained.

9.9-2 Modular Block Wall Unit

The following material specification for concrete segmental retaining wall (or MBW)units is from the *National Concrete Masonry Association*, Design Manual for Segmental Retaining Walls (Simac et al., 1993).

Section ________
SEGMENTAL RETAINING WALL UNITS

PART 1: GENERAL

1.01 Description

A. Work includes furnishing and installing segmental retaining wall (SRW) units to the lines and grades designated on the construction drawings or as directed by the Architect/Engineer. Also included is furnishing and installing appurtenant materials required for construction of the retaining wall as shown on the construction drawings.

1.02 Related Work

A. Section _____ - Site Preparation
B. Section _____ - Earthwork
C. Section _____ - Drainage Aggregate
D. Section _____ - Geosynthetic Reinforcement *{delete if not applicable}*

1.03 Reference Standards

A. ASTM C 90 - Load Bearing Concrete Masonry Units
B. ASTM C 140 - Sampling and Testing Concrete Masonry Units

C. ASTM D 698 - Moisture Density Relationship for Soils, Standard Method
D. NCMA TEK 50A - Specifications for Segmental Retaining Wall Units
E. NCMA SRWU-1 - Determination of Connection Strength between Geosynthetics and Segmental Concrete Units
F. NCMA SRWU-2 - Determination of Shear Strength between Segmental Concrete Units
G. NCMA Design Manual for Segmental Retaining Walls
H. Where specifications and reference documents conflict, the Architect/Engineer shall make the final determination of applicable document.

1.04 Certification

A. Contractor shall submit a notarized manufacturer's certificate prior to start of work stating that the SRW units meet the requirements of this specification.

1.05 Delivery, Storage and Handling

A. Contractor shall check the materials upon delivery to assure that specified type, grade, color and texture of SRW unit has been received.
B. Contractor shall prevent excessive mud, wet concrete, epoxies, and like material which may affix themselves from coming in contact with the materials.
C. Contractor shall protect the materials from damage. Damaged material shall not be incorporated into the reinforced soil wall.

1.06 Measurement and Payment

A. Measurement of SRW units is on a vertical square foot basis.
B. Payment shall cover supply and installation of SRW units along with appurtenant and incidental materials required for construction of the retaining wall as shown on the construction drawings. It shall include all compensation for labor, materials, supplies, equipment and permits associated with building these walls.
C. Quantity of retaining wall as shown on plans may be increased or decreased at the direction of the Architect/Engineer based on construction procedures and actual site conditions.
D. The accepted quantities of SRW units will be paid for per vertical square foot in place (total wall height). Payment will be made under:

Pay Item	Pay Unit
Segmental Retaining Wall Units	SQ FT

PART 2: MATERIALS

2.01 Segmental Retaining Wall Units

A. SRW units shall be machine formed concrete blocks specifically designed for retaining wall applications.
B. SRW units shall meet the following architectural requirements:
   1. Color of units shall be _____. *{insert}*
   2. Finish of units shall be _____. *{insert split-faced, smooth, striated, etc.}*
   3. Unit faces shall be of _____ geometry. *{insert rounded, straight, offset, etc.}*

4. Maximum and minimum face area per unit shall be _____ and _____, respectively. *{insert values or delete if not a requirement}*
5. Units shall be erected with a _____ configuration. *{insert running or stacked bond}*
6. All units shall be sound and free of cracks or other defects that would interfere with the proper placing of the unit or significantly impair the strength or permanence of the construction. Cracking and excessive chipping may be grounds for rejection.

C. SRW units shall meet the following structural requirements:
1. Concrete used to manufacture SRW units shall have a minimum 28 day compressive strength of *{3000 psi}* in accordance with ASTM C 90. The concrete shall have adequate freeze/thaw protection with a maximum moisture absorption rate, by weight, of: i) 8% in southern climates; or ii) 6% in northern climates.
2. Units shall be positively interlocked to provide a minimum shear capacity of a_U = *{400 lb/ft}* and λ_U = *{30 °}* as tested in accordance with NCMA SRWU-1.
3. Units shall provide a minimum connection strength between it and the geosynthetic reinforcement of a_{CS} = *{200 lb/ft}* and λ_{CS} = *{40 °}* as tested in accordance with NCMA SRWU-2, if required.
4. SRW units molded dimensions shall not differ more than ±1/8 inch from that specified, except height which shall be ±1/16 inch.

D. SRW units shall meet the following constructability and geometric requirements:
1. Units shall be capable of attaining concave and convex curves.
2. Units shall be positively engaged to the unit below so as to provide a minimum of 3/32 inch horizontal setback per vertical foot of wall height. True vertical stacked units will not be permitted.
3. Units shall be positively engaged to the unit below so as to provide a maximum of _____ inch horizontal setback per vertical foot of wall height. *{This controls amount of useable property at top of wall. Specify value or delete if not applicable.}*

2.02 Leveling Pad and Unit Fill Material

A. Material for footing shall consist of compacted sand or gravel and shall be a minimum of 6 inches in depth.
B. Fill for units shall be the free draining gravel or drainage fill, see Section _____ Drainage Aggregate.
C. Do not run mechanical vibrating plate compactors on top of the units. Compact unit fill be running hand-operated compaction equipment just behind the unit. Compact to minimum 95% standard Proctor density (ASTM D 698) or 90% of modified Proctor density (ASTM D 1557).

2.03 Drainage Aggregate

A. Drainage layer materials shall be the free draining gravel or drainage fill, see Section _____ - Drainage Aggregate.
B. Vertical drainage layer behind the wall face shall be placed no less than 1 ft^3 per 1 ft^2 of wall face.

2.04 Infill Soil

A. The infill soil material shall be free of debris and consist of either of the following inorganic soil types according to their USCS designations (GP, GW, SW, SP, SM, ML, CL). The maximum particle size shall be 4 inches. There shall be less than 20% by weight of particles greater than 1½ inches, maximum 60% by weight passing the #200 sieve and PI<20.
B. The infill soil shall be compacted in maximum 8 inches compacted lifts to the following minimum densities

(percentage of the maximum standard Proctor) (ASTM D 698): i) fine grained (ML-CL, SC, SM) soils to a minimum or 95%; and ii) coarse grained (GP, GW, SW, SP) soils to a minimum of 98%.

2.05 Common Backfill

A. Soil placed behind the infill can be any inorganic soil with a liquid limit less than 50 and plasticity index less than 30, or as directed by the Engineer.
B. Backfill shall be compacted to a minimum 90% of maximum standard Proctor density (ASTM D 698).

PART 3: EXECUTION

3.01 Excavation

A. Contractor shall excavate to the lines and grades shown on the project grading plans. Contractor shall take precautions to minimize over-excavation. Over-excavation shall be filled with compacted infill material, or as directed by the Engineer/Architect, at the Contractor's expense.
B. Architect/Engineer will inspect the excavation and approve prior to placement of bearing pad material.
C. Excavation of deleterious soils and replacement with compacted infill material, as directed by the Architect/Engineer, will be paid for at the contract unit prices, see Section ____ - Excavation..
D. Over-excavated areas in front of wall face shall be filled with compacted infill material at the Contractor's expense, or as directed by the Architect/Engineer.
E. Contractor shall verify location of existing structures and utilities prior to excavation. Contractor shall ensure all surrounding structures are protected from the effects of wall excavation.

3.02 Leveling Pad Construction

A. Leveling pad shall be placed as shown on the construction drawings with a minimum thickness of 6 inches.
B. Foundation soil shall be proofrolled and compacted to 95% of standard Proctor density and inspected by the Architect/Engineer prior to placement of leveling pad materials.
C. Soil leveling pad material shall be compacted to provide a level hard surface on which to place the first course of units. Compaction will be with mechanical plate compactors to 95% of maximum Proctor density (ASTM D 698).
D. Leveling pad shall be prepared to insure intimate contact of retaining wall unit with pad.

3.03 Segmental Unit Installation

A. First course of SRW units shall be placed on the bearing pad. The units shall be checked for level and alignment. The first course is the most important to insure accurate and acceptable results.
B. Insure that units are in full contact with base.
C. Units are placed side by side for full length of straight wall alignment. Alignment may be done by means of a string line or offset from base line to a molded finished face of the SRW unit. Adjust unit spacing for curved sections according to manufacturer's recommendations.
D. Install shear connectors (if applicable).
E. Place unit fill (if applicable).
F. Place and compact fill behind and within units.
G. Clean all excess debris from top of units and install next course. Ensure each course is completely filled prior to proceeding to next course.

H. Lay each successive course ensuring that shear connectors are engaged.

I. Repeat procedures to the extent of the wall height.

J. Uppermost row of SRW units or caps shall be glued to underlying units with an adhesive, as recommenced by the manufacturer.

END OF SECTION

9.9-3 Geosynthetic Wrap Around Wall

The following example of a special provision for materials and construction of permanent and temporary geosynthetic walls was obtained from the Washington State DOT

PERMANENT AND TEMPORARY GEOSYNTHETIC WALLS

1997

1.0 Description

The Contractor shall furnish and construct temporary or permanent geosynthetic retaining walls in accordance with the details shown in the Plans, these specifications, or as directed by the Engineer.

2.0 Materials

2.1 Geosynthetic Storage and Handling

Geosynthetic roll identification, storage, and handling shall be in conformance to ASTM D 4873. During periods of shipment and storage, the geosynthetic shall be stored off the ground. The geosynthetic shall be covered at all times during shipment and storage such that it is fully protected from ultraviolet radiation including sunlight, site construction damage, precipitation, chemicals that are strong acids or strong bases, flames including welding sparks, temperatures in excess of 70° C, and any other environmental condition that may damage the physical property values of the geosynthetic.

2.2 Borrow for Geosynthetic Retaining Wall

All backfill material used in the reinforced soil zone of the geosynthetic wall shall conform to the requirements of Gravel Borrow which are as follows:

Sieve Size	Percent Passing
31.5 mm	100
4.75 mm	50-80
0.425 mm	30 max.
0.075 mm	7.0 max.
Sand Equivalent	45 min.

All percentages are by mass.

Gravel Borrow shall be either naturally occurring or processed, and shall be free draining, free from organic or otherwise deleterious material. The material shall be substantially free of shale or other soft, poor durability particles, and shall not contain recycled materials, such as glass, shredded tires, portland cement concrete rubble, or asphaltic concrete rubble. The backfill material shall meet the following requirements:

Property	Test Method	Allowable Test Value
Los Angeles Wear, 500 rev.	AASHTO T 96	35 percent max.
Degradation	WSDOT Test Method 113	15 min.
pH	AASHTO T 289-91	Quantity **

** 4.5 to 9 for permanent walls and 3 to 10 for temporary walls

Wall backfill material satisfying these gradation, durability and chemical requirements shall be classified as nonaggressive.

2.3 Geosynthetic and Thread for Sewing

The term geosynthetics shall include both geotextiles and geogrids.

Geotextiles shall consist of only of long chain polymeric fibers or yarns formed into a stable network such that the fibers or yarns retain their position relative to each other during handling, placement, and design service life. At least 95 percent by weight of the material shall be polyolefins or polyesters. The material shall be free from defects or tears. The geotextile shall also be free of any treatment or coating which might adversely alter its hydraulic or physical properties after installation.

Geotextile reinforcement in geosynthetic retaining walls shall conform to the properties specified in Tables 1 and 2 for permanent walls, and Tables 1 and 3 for temporary walls.
Geogrids shall consist of a regular network of integrally connected polymer tensile elements with an aperture geometry sufficient to permit mechanical interlock with the surrounding backfill. The long chain polymers in the geogrid tensile elements, not including coatings, shall consist of at least 95 percent by mass of the material of polyolefins or polyesters. The material shall be free of defects, cuts, and tears. Geogrid reinforcement in geosynthetic retaining walls shall conform to the properties specified in Table 2 for permanent walls, and Table 3 for temporary walls.

For geosynthetic walls which use geogrid reinforcement, the geotextile material placed at the wall face to retain the backfill material as shown in the Plans shall conform to the properties for Construction Geotextile for Underground Drainage, Moderate Survivability, Class A.

Thread used for sewing shall consist of high strength polypropylene, polyester, or polyamide. Nylon threads will not be allowed. The thread used to sew geotextile seams in exposed wall faces shall be resistant to ultraviolet radiation. The thread shall be of contrasting color to that of the geotextile itself.

2.4 Geosynthetic Properties

2.4.1 Geosynthetic Properties For Retaining Walls

The requirements of this subsection apply to both permanent and temporary walls.

All geotextile properties provided in Table 1 are minimum average roll values. The average test results for any sampled roll in a lot shall meet or exceed the values shown in the table. The test procedures specified in the table are in conformance with the most recently approved ASTM geotextile test procedures, except for geotextile sampling and specimen conditioning, which are in accordance with WSDOT Test Methods 914 and 915, respectively.

Table 1: Minimum properties required for geotextile reinforcement used in geosynthetic reinforced slopes and walls.

Geotextile Property	Test Method	Geotextile Property Requirements Woven/Nonwoven
Water Permittivity	ASTM D4491	.02 $sec.^{-1}$ min.
AOS	ASTM D4751	.84 mm max.
Grab Tensile Strength, min. in machine and x-machine direction	ASTM D4632	900 N/530 N min.
Grab Failure Strain, in machine and x-machine direction	ASTM D4632	< 50% / ≥ 50%
Seam Breaking Strength[1]	ASTM D4632	700 N/430 N min.
Puncture Resistance	ASTM D4833	280 N/220 N min.
Tear Strength, min. in machine and x-machine direction	ASTM D4533	280 N/220 N min.
Ultraviolet (UV) Radiation Stability	ASTM D4355	70% (for polypropylene and polyethylene) and 50% (for polyester) Strength Retained min., after 500 Hr. in weatherometer

[1]Applies only to seams perpendicular to the wall face.

For geogrids, the summation of the geogrid joint strengths determined in accordance with Geosynthetic Research Institute test method GRI:GG2 occurring in a 300 mm length of grid in the direction of loading (i.e., perpendicular to the wall face) shall be greater than or equal to the ultimate strength (T_{ult}) of the grid element to which they are attached. For this determination, T_{ult} is to be determined using Geosynthetic Research Institute test method GRI:GG1. If the joint spacing is greater than or equal to 300 mm, two joints shall be included in this summation of joint strengths. The ultraviolet (UV) radiation stability, ASTM D4355, shall be a minimum of 70% strength retained after 500 hours in the weatherometer for polypropylene and polyethylene geogrids, and 50% strength retained after 500 hours in the weatherometer for polyester geogrids.

2.4.2 Geosynthetic Properties for Permanent Retaining Walls

Table 2: Long-term tensile strength, T_{al}, required for the geosynthetic reinforcement used in geosynthetic retaining walls.

[3]Wall Location	Vertical Spacing of Reinforcement Layers	Reinforcement Layer Distance from Top of Wall	[1,2,3]Minimum Long-Term Tensile Strength, T_{al}
$$1$$	***$$2$$***	***$$3$$***	***$$4$$***

[1]These long-term tensile strength requirements apply only in the geosynthetic direction perpendicular to the wall face.

[2]T_{al} shall be determined in accordance with WSDOT Test Method 925, "Determination of Long-Term Strength for Geosynthetic Reinforcement."

[3]Walls ***$$5$$*** are classified as Class ***$$6$$*** structures.

2.4.3 Geosynthetic Properties For Temporary Retaining Wall

Wide strip geosynthetic strengths provided in Table 9 are minimum average roll values. The average test results for any sampled roll in a lot shall meet or exceed the values shown in the table. These wide strip strength requirements apply only in the geosynthetic direction perpendicular to the wall face. The test procedures specified in the table are in conformance with the most recently approved ASTM geosynthetic test procedures, except for geosynthetic sampling and specimen conditioning, which are in accordance with WSDOT Test Methods 914 and 915, respectively.

Table 3: Wide strip tensile strength required for the geosynthetic reinforcement used in geosynthetic retaining walls.

Wall Location	Vertical Spacing of Reinforcement Layers	Reinforcement Layer Distance from Top of Wall	Minimum Tensile Strength Based on ASTM D4595
$$1$$	***$$2$$***	***$$3$$***	***$$4$$***

ASTM D4595 shall be modified to address geogrids as follows: The minimum specimen width shall be 200 mm with a minimum gauge length of 100 mm. The gauge length shall be a minimum of two grid apertures long. The gauge length shall be in increments of whole grid apertures. For the purpose of calculating tensile strength, the specimen width shall be considered to be the distance between the outermost ribs of the specimen as measured at the midpoint of those ribs plus the average center to center spacing between ribs.

2.5 Source Approval

2.5.1 Permanent Geosynthetic Retaining Wall

Geosynthetic products which are qualified for use in geosynthetic reinforced structures (Classes 1, 2, or both) are listed in the current Qualified Products List (QPL).

For geosynthetic products proposed for use which are not listed in the current QPL, the Contractor shall submit test information and the calculations used in the determination of T_{al} performed in accordance with WSDOT Test Method 925 to the Olympia Service Center Materials Laboratory in Tumwater for evaluation. The Contracting Agency will require up to 30 calendar days after receipt of the information to complete the evaluation.

Source approval for retaining wall geosynthetic materials listed in the current QPL, or as approved based on data developed and submitted in accordance with WSDOT Test Method 925, will be based on conformance to the applicable values in Tables 1 and 2.

2.5.2 Temporary Geosynthetic Retaining Wall

The Contractor shall submit to the Engineer the following information regarding each geosynthetic proposed for use:

Manufacturer's name and current address,
Full product name,
Geosynthetic structure, including fiber/yarn type, and
Geosynthetic polymer type(s).

If the geosynthetic source has not been previously evaluated or included in the QPL, a sample of each proposed geosynthetic shall be submitted to the Olympia Service Center Materials Laboratory in Tumwater for evaluation and testing. A maximum of 14 calendar days will be required for this testing once the samples and required product information arrive at the Materials Laboratory. Source approval will be based on conformance to the applicable values in Tables 1 and 3. Source approval will not be the basis of acceptance of specific lots of material unless the lot sampled can be clearly identified, and the number of samples tested and approved meet the requirements of WSDOT Test Method 914.

Each sample shall have minimum dimensions of 1.5 meters by the full roll width of the geosynthetic. A minimum of 6 square meters of geosynthetic shall be submitted to the Engineer for testing. The geosynthetic machine direction shall be marked clearly on each sample submitted for testing. The machine direction is defined as the direction perpendicular to the axis of the geosynthetic roll.

The geosynthetic samples shall be cut from the geosynthetic roll with scissors, sharp knife, or other suitable method which produces a smooth geosynthetic edge and does not cause geosynthetic ripping or tearing. The samples shall not be taken from the outer wrap of the geosynthetic roll nor the inner wrap of the core.

2.6 Acceptance Samples

Samples will be randomly taken by the Engineer at the job site to confirm that the geosynthetic meets the property values specified.

Approval will be based on testing of samples from each lot. A "lot" shall be defined for the purposes of this specification as all geosynthetic rolls within the consignment (i.e., all rolls sent the project site) which were produced by the same manufacturer during a continuous period of production at the same manufacturing plant and have the same product name. After the samples have arrived at the Olympia Service Center Materials Laboratory in Tumwater, a maximum of 14 calendar days will be required for this testing.

2.6.1 Permanent Geosynthetic Retaining Wall

Geotextile acceptance testing shall meet the requirements of Table 1, and both geotextile and geogrid acceptance testing shall meet the required ultimate tensile strength T_{ult} as provided in the QPL for the selected product(s). If the selected product(s) are not listed in the current QPL, the result of the testing for T_{ult} must be greater than or equal to T_{ult} as determined from the product data submitted and approved by the Olympia Service Center Materials Laboratory during source approval. If the results of the testing show that the retaining wall geosynthetic lot does not meet the specified properties, the roll or rolls which were sampled will be rejected. Two additional rolls for each roll tested which failed from the lot previously tested will then be selected at random by the Engineer for sampling and retesting. If the retesting shows that any of the additional rolls tested do not meet the required properties, the entire lot will be rejected. If the test results from all the rolls retested meet the required properties, the entire lot minus the roll(s) which failed will be accepted. All geosynthetic which has defects, deterioration, or damage, as determined by the Engineer, will also be rejected. All rejected geosynthetic shall be replaced at no expense to the Contracting Agency.

2.6.2 Temporary Geosynthetic Retaining Wall

If the results of the testing show that the retaining wall geosynthetic lot does not meet the properties specified in Table 1 (geotextiles only) and Table 3 (geotextiles and geogrids), the roll or rolls which were sampled will be rejected. Two additional rolls for each roll tested which failed from the lot previously tested will then be selected at random by the Engineer for sampling and retesting. If the retesting shows that any of the additional rolls tested do not meet the required properties, the entire lot will be rejected. If the test results from all the rolls retested meet the required properties, the entire lot minus the roll(s) which failed will be accepted. All geosynthetic which has defects, deterioration, or damage, as determined by the Engineer, will also be rejected. All rejected geosynthetic shall be replaced at no expense to the Contracting Agency.

2.6.3 Approval of Seams

If the geotextile seams are to be sewn in the field, the Contractor shall provide a section of sewn seam which can be sampled by the Engineer before the geotextile is installed.

The seam sewn for sampling shall be sewn using the same equipment and procedures as will be used to sew the production seams. The seams sewn for sampling must be at least 2 meters in length. If the seams are sewn in the factory, the Engineer will obtain samples of the factory seam at random from any of the rolls to be used. The seam assembly description shall be submitted by the Contractor to the Engineer and will be included with the seam sample obtained for testing. This description shall include the seam type, stitch type, sewing thread type(s), and stitch density.

2.7 Acceptance by Certificate of Compliance

The Contractor shall provide a Manufacturer's Certificate of Compliance to the Engineer for all quantities of retaining wall geosynthetic material.

The Manufacturer's Certificate of Compliance shall include the following information about each geosynthetic roll to be used:

Manufacturer's name and current address,
- Full product name,
- Geosynthetic structure, including fiber/yarn type,
- Geosynthetic polymer type(s),
- Geosynthetic roll number, and
- Certified test results.

3.0 Geosynthetic Wall Construction Requirements

3.1 Submittals

The Contractor shall submit to the Engineer, a minimum of 14 calendar days prior to beginning construction of each wall, detailed plans for each wall and as a minimum, the submittals shall include the following:

1. Detailed wall plans showing the actual lengths proposed for the geosynthetic reinforcing layers and the locations of each geosynthetic product proposed for use in each of the geosynthetic reinforcing layers.
2. The Contractor's proposed wall construction method, including proposed forming systems, types of equipment to be used and proposed erection sequence.
3. Manufacturer's Certificate of Compliance, samples of the retaining wall geosynthetic and sewn seams for the purpose of acceptance as specified.
4. Details of geosynthetic wall corner construction, including details of the positive connection between the wall sections on both sides of the corner.
5. Details of terminating a top layer of retaining wall geosynthetic and backfill due to a changing retaining wall profile.

Approval of the Contractor's proposed wall construction details and methods shall not relieve the Contractor of their responsibility to construct the walls in accordance with the requirements of these Specifications.

3.2 Wall Construction

The Contractor shall excavate for the retaining wall in accordance with Section 2-09, and conforming to the limits and construction stages shown in the Plans.

The Contractor shall direct all surface runoff from adjacent areas away from the retaining wall construction site.

The area to be covered by the geosynthetic shall be graded to a smooth, uniform condition free from ruts, potholes, and protruding objects such as rocks or sticks. The geosynthetic shall be spread immediately ahead of the covering operation. The geosynthetic shall not be left exposed to sunlight during installation for a total of more than 14 calendar days. The geosynthetic shall be laid smooth without excessive wrinkles.

The Contractor shall begin wall construction at the lowest portion of the excavation and shall place each layer horizontally as shown in the Plans. The Contractor shall complete each layer entirely before beginning the next layer.

Geotextile splices shall consist of a sewn seam or a minimum 300 mm overlap. Geogrid splices shall consist of adjacent geogrid strips butted together and fastened using hog rings, or other methods approved by the Engineer, in such a manner to prevent the splices from separating during geogrid installation and backfilling. Splices exposed at the wall face shall prevent loss of backfill material through the face. The splicing material exposed at the wall face shall be as durable and strong as the material to which the splices are tied. The Contractor shall offset geosynthetic splices in one layer from those in the other layers such that the splices shall not line up vertically. Splices parallel to the wall face will not be allowed, as shown in the Plans.

If geotextile seams are to be sewn in the field or at the factory, the seams shall consist of one row of stitching unless the geotextile where the seam is to be sewn does not have a selvage edge. If a selvage edge is not present, the seams shall consist of two parallel rows of stitching, or shall consist of a J-seam, Type SSn-1, using a single row of stitching. The two rows of stitching shall be 25 mm apart with a tolerance of plus or minus 13 mm and shall not cross except for restitching. The stitching shall be a lock-type stitch. The minimum seam allowance, i.e., the minimum distance from the geotextile edge to the stitch line nearest to that edge shall be 40 mm if a flat or prayer seam, Type SSa-2, is used. The minimum seam allowance for all other seam types shall be 25 mm. The seam, stitch type, and the equipment used to perform the stitching shall be as recommended by the manufacturer of the geotextile and as approved by the Engineer.

The seams shall be sewn in such a manner that the seam can be inspected readily by the Engineer or a representative. The seam strength will be tested and shall meet the requirements stated herein.

The Contractor shall stretch out the geosynthetic in the direction perpendicular to the wall face to ensure that no slack or wrinkles exist in the geosynthetic prior to backfilling.

For geogrids, the length of the reinforcement required as shown in the Plans shall be defined as the distance between the geosynthetic wrapped face and the last geogrid node at the end of the reinforcement in the wall backfill.

Under no circumstances shall be geosynthetic be dragged through mud or over sharp objects which could damage the geosynthetic. The Contractor shall place fill material on the geosynthetic in lifts such that 150 mm minimum of fill material is between the vehicle or equipment tires or tracks and the geosynthetic at all times. Construction vehicles shall be limited in size and weight, to reduce rutting in the initial lift above the geosynthetic, to not greater than 75 mm deep to prevent overstressing the geosynthetic. The Contractor shall remove all particles within the backfill material greater than 75 mm in size. Turning of vehicles on the first lift above the geosynthetic will not be permitted. The Contractor shall not end dump fill material directly on the geosynthetic without the prior approval of the Engineer.

Should the geosynthetic be torn, punctured, or the splices or sewn joints disturbed as evidenced by visible geosynthetic damage, subgrade pumping, intrusion, or distortion, the backfill around the damaged or displaced area shall be removed. The damaged geosynthetic section shall be replaced by the Contractor with a new section of geosynthetic at no expense to the Contracting Agency.

The Contractor shall use a temporary form system to prevent sagging of the geosynthetic facing elements during construction. A typical example of a temporary form system and sequence of wall construction required when using this form are detailed in the Plans. Soil piles or the geosynthetic manufacturer's recommended method, in combination with the forming system shall be used to hold the geosynthetic in place until the specified cover material is placed.

The Contractor shall place and compact the wall backfill in accordance with the wall construction sequence detailed in the Plans. The minimum compacted backfill lift thickness of the first lift above each geosynthetic layer shall be 150 mm. The maximum compacted lift thickness anywhere within the wall shall be 250 mm.

The Contractor shall compact each layer to 95 percent of maximum density. The water content of the wall backfill shall not exceed the optimum water content by more than 3 percent. The Contractor shall not use sheepsfoot rollers or rollers with protrusions. Rollers which have a mass of more than 2,700 Kg shall be used with the vibrator turned off. The Contractor may use rollers which have a mass of 2,700 Kg or less with the vibrator turned on with the prior approval of the Engineer. The Contractor shall compact the zone within 1 meter of the wall face without causing damage or distortion to the wall facing elements or reinforcing layers by using light mechanical tampers approved by the Engineer.

The Contractor shall construct wall corners at the locations shown in the Plans, and in accordance with the wall corner construction sequence and method submitted by the Contractor and approved by the Engineer. Wall angle points with an interior angle of less than 150 degrees shall be considered to be a wall corner. The wall corner shall provide a positive connection between the sections of the wall on each side of the corner such that the wall backfill material cannot spill out through the corner at any time during the design life of the wall. The Contractor shall construct the wall corner such that the wall sections on both sides of the corner attain the full geosynthetic layer embedment lengths shown in the Plans.

Where required by retaining wall profile grade, the Contractor shall terminate top layers of retaining wall geosynthetic and backfill in accordance with the method submitted by the Contractor and approved by the Engineer. The end of each layer at the top of the wall shall be constructed in a manner which prevents wall backfill material from spilling out the face of the wall throughout the life of the wall. If the profile of the top of the wall changes at a rate of 1:1 or steeper, this change in top of wall profile shall be considered to be a corner.

3.3 Tolerances
The Contractor shall complete the base of the retaining wall excavation to within plus or minus 75 mm of the staked elevations unless otherwise directed by the Engineer. The Contractor shall place the external wall dimensions to within plus or minus 50 mm of that staked on the ground. The Contractor shall space the reinforcement layers vertically and place the overlaps to within plus or minus 25 mm of that shown in the Plans.

The completed wall(s) shall meet the following tolerances:

	Permanent Wall	Temporary Wall
Deviation from the design batter and horizontal alignment for the face when measured along a 3 meter straight edge at the midpoint of each wall layer shall not exceed:	75 mm	130 mm
Deviation from the overall design batter per 3 meters of wall height shall not exceed:	50 mm	75 mm
Maximum outward bulge of the face between backfill reinforcement layers shall not exceed:	100 mm	150 mm

3.4 Permanent Facing for Geosynthetic Retaining Wall
The Contractor shall apply a permanent wall facing to the surface of the geosynthetic retaining wall as shown in the Plans.

9.10 CONSTRUCTION PROCEDURES

9.10-1 Concrete Faced Wall

The construction of a geosynthetic MSE system with precast facing elements requires the following steps.

A. Prepare Subgrade

1. The foundation should be excavated to the grade as shown on the plans.

2. The excavated areas should be carefully inspected. Any unsuitable foundation soils should be compacted or excavated and replaced with compacted select backfill material.

3. Foundation soil at the base of the wall excavation should be proofrolled with a vibratory or rubber-tired roller.

B. Leveling Pad

1. A cast-in-place or precast concrete leveling pad should be placed at the foundation elevation for all MSE structures with concrete (panel and MBW unit) facing elements. The unreinforced concrete pad is often only 0.3 m wide and 0.15 m thick. The purpose of the pad is to serve as a guide for facing panel erection and not to act as a structural foundation support.

C. Erection of Facing Units

1. The first row of facing panels may be full- or half-height panels, depending upon the type of facing utilized. The first tier of panels must be shored up to maintain stability and alignment. For construction with MBW units, full-sized blocks are used throughout, with no shoring required.

2. Erection of subsequent rows of facing panels proceed incremental with fill placement and compaction.

D. Backfill Placement and Compaction

1. The backfill material should be placed over a compacted lift thickness as specified.

2. The backfill material should be compacted to specified density, usually 95 to 100% of AASHTO T-99 maximum density.

3. A key to good performance is **consistent** compaction. Wall fill lift thickness must be controlled, based upon specification requirements and vertical distribution of reinforcement elements (and incremental face unit height).

E. Reinforcement Placement

1. The geosynthetic reinforcements are placed and connected to the facing units when the fill has been brought up to the level of the connection. Reinforcement is generally placed perpendicular to the back of the facing units.

F. Placement of Fill on Reinforcement

1. Geosynthetic reinforcement shall be pulled taut and anchored prior to placing fill.

2. Fill placement and spreading should prevent or minimize formation of wrinkles in the geosynthetic. Wrinkles or slack near the facing connection should be prevented as they can result in differential movement of the wall face.

3. A minimum thickness of 150 mm of fill must be maintained between the tracks of the construction equipment and the reinforcement at all times.

9.10-2 Geotextile Wrap-around Wall

Construction procedures for geosynthetic MSE walls are straightforward. Experience of the U.S. Forest Service, the New York Department of Transportation, and the Colorado Department of Highways has been very valuable in developing technically feasible and economical construction procedures. These procedures, detailed in Christopher and Holtz (1985), as well as the other references, are outlined below.

A. Wall Foundation

1. The foundation should be excavated to the grade as shown on the plans. It should be graded level for a width equal to the length of reinforcement plus 0.3 m.

2. The excavated areas should be carefully inspected. Any unsuitable foundation soils should be compacted or excavated and replaced with compacted select backfill material.

3. Foundation soil at the base of the wall excavation should be proofrolled with vibratory or rubber-tired roller.

B. Placement of geosynthetic reinforcement

1. The geosynthetic should be placed with the principal strength (machine) direction perpendicular to the face of the wall. It should be secured in place to prevent movement during fill placement.

2. It may be more convenient to unroll the geosynthetic with the machine direction parallel to the wall alignment. If this is done, then the cross-machine design tensile strength must be greater or equal to the design tensile strength.

3. A minimum of 150 mm overlap is recommended along edges parallel to the reinforcement for wrapped-faced walls.

4. If large foundation settlements are anticipated, which might result in separation between overlap layers, then field-sewing of adjacent geotextile sheets is recommended. Geogrids should be mechanically fastened in that direction.

C. Backfill placement in reinforced section

1. The backfill material should be placed over the reinforcement with a compacted lift thickness of 200 mm or as determined by the Engineer.

2. The backfill material should be compacted to at least 90% of the ASTM D 1557 or AASHTO T-99 Standard Proctor maximum density at or below the optimum moisture. Alternatively, a relative density compaction specification could be used. For coarse, gravelly backfills, a method-type compaction specification is appropriate.

3. When placing and compacting the backfill material, avoid any folding or movement of the geosynthetic.

4. A minimum thickness of 150 mm of fill must be maintained between the wheels of the construction equipment and the reinforcement at all times.

D. Face construction and connections

1. Place the geosynthetic layers using face forms as shown in Figure 9-8, unless a precast propped panel facing is to be used.

2. When using temporary support of forms at the face to allow compaction of the backfill against the wall face, form holders should be placed at the base of each layer at 1 m horizontal intervals. Details of temporary form work for geosynthetics are shown in Figure 9-8.

3. When using geogrids, it may be necessary to use a geotextile or wire mesh to retain the backfill material at the wall face (Figure 9-9).

4. A hand-operated vibratory compactor is recommended when compacting backfill within 0.6 m of the wall face.

5. The return-type method shown in Figure 9-4a can be used for facing support. The geosynthetic is folded at the face over the backfill material, with a minimum return length of 1 m to ensure adequate pullout resistance.

6. Apply facing treatment (shotcrete, precast facing panels, etc.). Figure 9-4 shows several facing alternatives for geosynthetic walls.

9.11 INSPECTION

As with all geosynthetic construction, and especially with critical structures such as MSE walls and abutments, competent and professional construction inspection is absolutely essential for a successful project. The Engineer should develop procedures to ensure that the specified material is delivered to the project, that the geosynthetic is not damaged during construction, and that the specified sequence of construction operations are explicitly followed. Inspectors should use the checklist in Section 1.7. Other important details include construction of the wall face and application of the facing treatment to minimize the geosynthetic's exposure to ultraviolet light.

Because geosynthetic MSE retaining walls are sometimes considered *experimental*, they often are instrumented. In these cases, as a minimum, settlements and outward movements of the wall at its top should be determined by ordinary levels and triangulation surveys. Sometimes inclinometers and/or multiple-point extensometers are used for observing potential horizontal movements. On major projects, strain gages are placed on the geosynthetic to measure internal strains (Allen et al., 1991).

9.12 IMPLEMENTATION

There are two primary issues regarding implementation of geosynthetic MSE wall technology within a transportation agency. They are: determining who will complete and be responsible for final design; and specifying determination of the allowable and design tensile strengths of geosynthetic reinforcement.

MSE walls may be contracted using two different approaches. MSE walls can be contracted on the basis of:

- in-house (agency) design with geosynthetic reinforcement, facing, drainage, and construction execution specified; or
- system or end-result design approach using approved systems with lines and grades noted on the drawings.

Both options are acceptable, but of course, the in-house design approach is the preferred engineering approach. The in-house option will enable agency engineers to examine more facing and reinforcement options during design. This option requires engineering staff trained in MSE technology. Design responsibility is, however, well defined. This trained staff would be valuable during construction, when questions and/or design modification requests arise.

The end-result approach, with sound specifications and prequalification of suppliers and materials, may offer some benefits. Design of the MSE structure is completed by staff experienced with the specific system, though not necessarily experienced with local soil and construction conditions. The prequalified material components of geosynthetic and facing units have been successfully and routinely used together. While the system specification approach lessens the engineering requirements for an agency, and transfers some of a project's design cost to construction, design responsibility must be clearly established: In this case, the designer does not work for the owner!

Another issue difficult for many agencies to address is the evaluation and specification of the allowable and design tensile strengths of geosynthetic reinforcement. The procedure for evaluation, as summarized within this chapter and in previous chapters on reinforced slopes, is detailed in Appendix K.

This recommended procedure is based upon the assumption that materials will be prequalified and listed on an *approved products* list as specifications. The recommended requirements for supplier submissions and for agency review, and recommended delineation of responsibilities within a typical agency, are presented in the FHWA guidelines (Berg, 1993). This procedure has been cumbersome for agencies that do not use approved products lists or that review and approve products based upon specific project submittals.

Because of these implementation problems, an alternative starting point procedure for determining long-term allowable design strength of geosynthetic soil reinforcing elements has been prepared and is presented in Appendix K. This new, proposed procedure is meant to complement, and not supersede, the recommended procedure of detailed testing and evaluation of geosynthetic reinforcement materials.

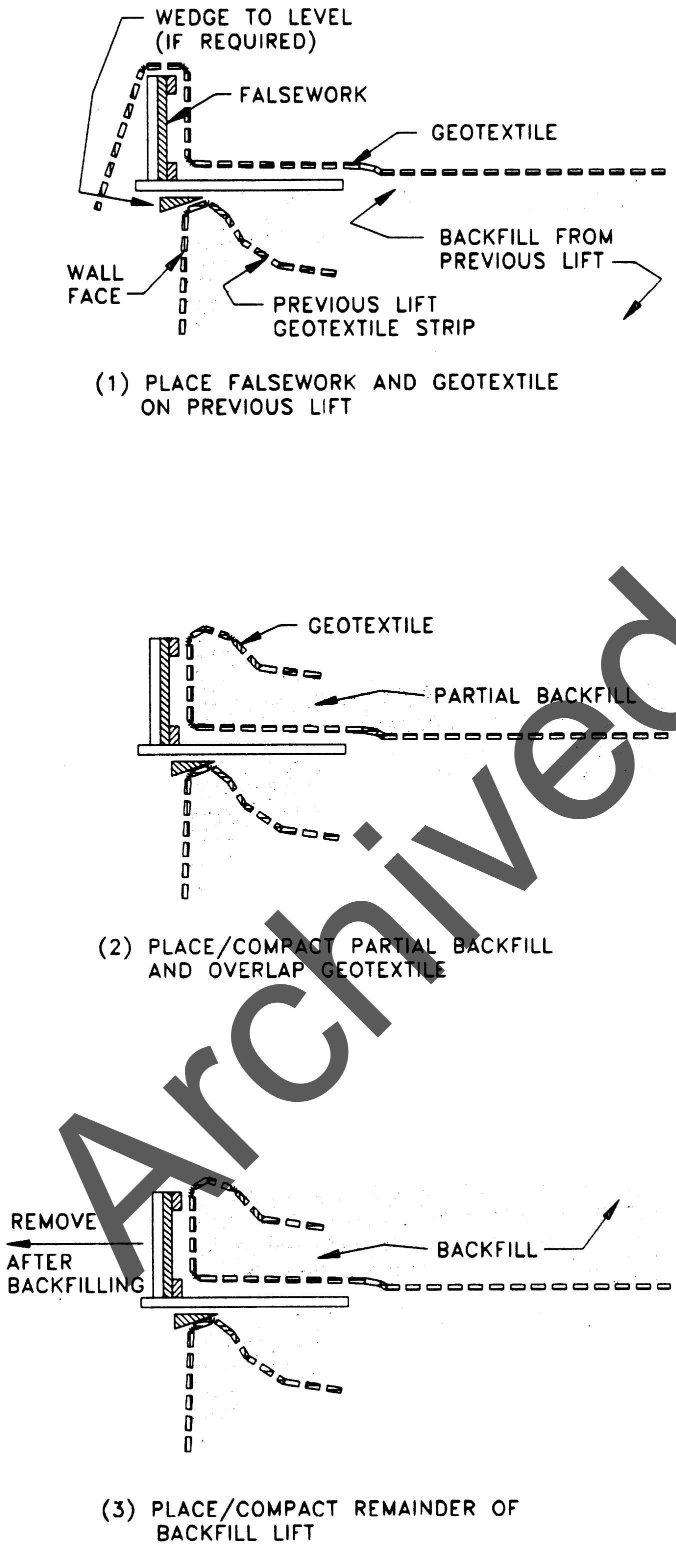


Figure 9-8 Lift construction sequence for geotextile reinforced soil walls (Steward, et al., 1977).

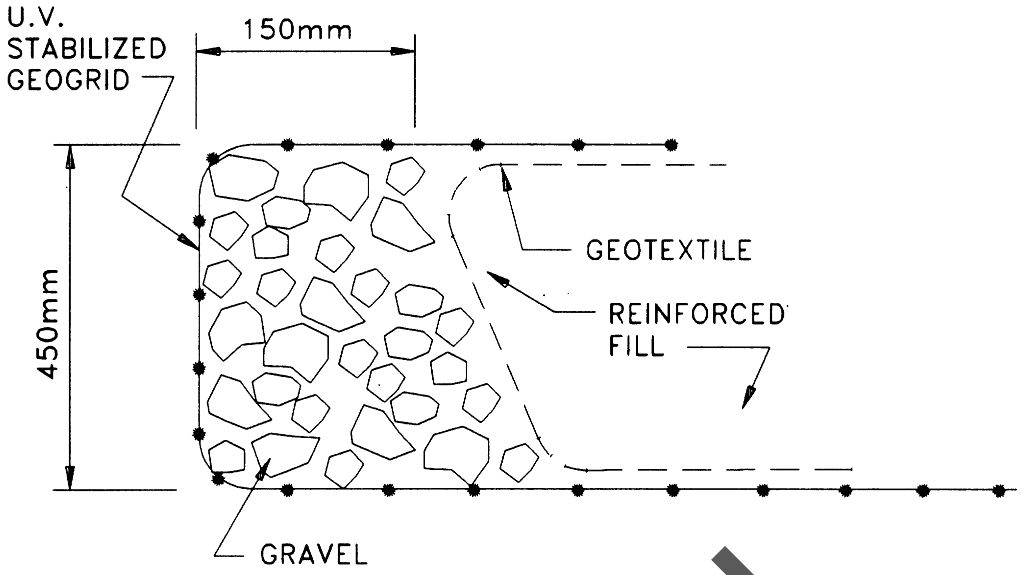


Figure 9-9 Typical face construction detail for vertical geogrid-reinforced retaining wall faces.

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10.0 GEOMEMBRANES AND OTHER GEOSYNTHETIC BARRIERS

10.1 BACKGROUND

Barriers are used in earthwork construction to control movement of water, other liquids and sometimes vapors. Barriers are used to waterproof structures, to prevent moisture changes beneath roadways, to contain water and wastes, and to support other applications in transportation works. The function of these barriers is to either prevent damage to highway pavements and structures or to contain water or waste materials. Barriers must be engineered to perform their intended function for the particular application and project being designed.

Traditional barriers, or liners, are field-constructed of soil or aggregate-based materials. Thick compacted clay layers, cast-in-place concrete, and asphalt concrete are used to construct liners. Another conventional liner material is geomembranes, which have been used in transportation applications for more than forty years. The U.S. Bureau of Reclamation has been using geomembranes in water conveyance canals since the 1950s (Staff, 1984). Other types of geosynthetic barriers have also been used in transportation applications. These include thin-film geotextile composites, geosynthetic clay liners, and field-impregnated geotextiles.

While soil or aggregate-based liners are well-suited to some applications, geomembrane and other geosynthetic barriers are more appropriate for other projects. Suitability may be defined during design and with due consideration to material availability, long-term performance, and cost. For example, rigid concrete and asphalt liners or semi-stiff compacted clay liners are not well-suited to sites where barriers are subject to foundation settlements; conversely a geomembrane which has adequate flexibility would be suitable.

10.2 GEOSYNTHETIC BARRIER MATERIALS

Geosynthetic barrier materials can be classified as geomembranes, thin-film geotextile composites, geosynthetic clay liners, or field-impregnated geotextiles. Materials within each classification are reviewed herein, starting with a general material definition. The components, manufacturing processes, resultant product characteristics, and typical dimensions are presented. Available test standards used to quantify property values are listed in Appendix E.

10.2-1 Geomembranes

The term *geomembrane* is defined as a very low-permeability synthetic membrane liner or barrier used with any geotechnical engineering-related material to control fluid migration in a man-made project, system, or structure (ASTM D 4439). However, within this document the term geomembrane will be used to specifically describe materials which are manufactured of continuous polymeric sheets. Commonly available geomembranes are manufactured of the polymers listed in Table 10-1.

TABLE 10-1
COMMON TYPES OF GEOMEMBRANES

POLYMER TYPE	Abbreviation	Available	
		without scrim reinforcement	with scrim reinforcement
Chlorinated Polyethylene	CPE	✔	✔
Chlorosulfonated Polyethylene	CSPE	✔	✔
Ethylene Interpolymer Alloy	EIA		✔
High-Density Polyethylene	HDPE	✔	
Polypropylene	PP	✔	✔
Polyvinyl Chloride	PVC	✔	✔
Linear Low-Density Polyethylene	LLDPE	✔	

The HDPE and LLDPE geomembranes are supplied in roll form. Widths of approximately 4.6 to 10.5 m are available. Roll lengths of 200 to 300 m are typical, although custom roll lengths are available. These materials are generally available in sheet thicknesses of 1.0, 1.5, 2.0, and 2.5 mm. These PE geomembranes are usually of singular (*i.e.*, not composite) manufacture. However, thick coextruded PE composites are available with light-colored heat reflective surfaces, electrical conductive surfaces (for leakage testing), or a LLDPE layer sandwiched between an upper and lower layer of HDPE.

Geomembranes manufactured of CPE, CSPE, PVC, and PP are supplied in large panels, accordion-folded onto pallets. The panels are fabricated by factory-seaming of rolls, typically 1.4 to 2.5 m in width. Panels as large as 1,800 m^2 are available. These geomembranes may be of singular manufacture or composites with a fabric scrim incorporated to modify the mechanical properties (Table 10-2).

Geomembranes are *relatively* impermeable materials - *i.e.*, all materials are permeable, but the permeability of geomembranes (on the order of 10^{-14} m/s) is significantly lower than that of compacted clays. Hence, geomembranes are sometimes referred to as being *impermeable*, relative to soil. Theoretically, multiple layers of geomembranes and drains can be utilized to construct impermeable structures (Giroud, 1984).

Leakage, and not permeability, is the primary concern when designing geomembrane containment structures. Leakage can occur through poor field seams, poor factory seams, pinholes from manufacture, and puncture holes from handling, placement, or in-service loads. Leakage of geomembrane liner systems is minimized by attention to design, specification, testing, quality control (QC), quality assurance (QA) of manufacture, and QC and QA of construction.

Geomembrane materials are better-defined than other geosynthetic barriers, due to their widespread use in environmental applications and their historical use in other applications. Manufacturing QC/QA standards, index property test methods, performance test methods, design requirements, design detailing, and construction QC/QA are well-established (Daniel and Koerner, 1993).

10.2-2 Thin-Film Geotextile Composites

The moisture barrier commonly used in roadway reconstruction is a thin-film geotextile composite. These composites are used to prevent or minimize moisture changes in pavement subgrades, as discussed in Section 10.3. Two styles of composites, as illustrated in Figure 10-1, are available.

One commercially available composite consists of a very lightweight PP nonwoven geotextile sandwiched between two layers of PP film (Figure 10-1a). This product has a mass per unit area of 140 g/m^2 and is available in roll widths of 2.3 m and roll lengths of 91 m.

Another commercially available composite consists of a polyethylene (PE) film sandwiched between two layers of nonwoven PP geotextiles (Figure 10-1b). This product has a mass per unit area of 300 g/m^2 and is available in roll widths of 3.65 m and roll lengths of 91 m. Similar products are available with PVC, CSPE, and PP geomembranes as the core.

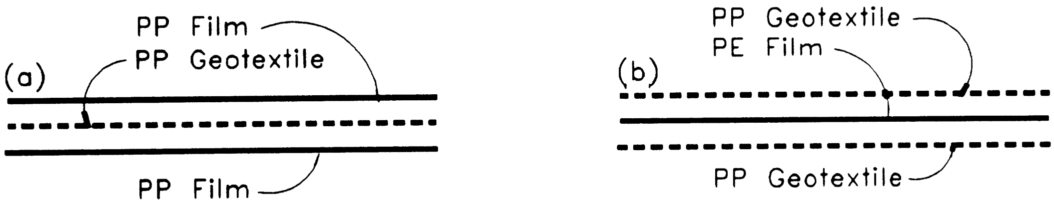

Figure 10-1 Thin-film geotextile composites: *(a)* PP film / PP geotextile / PP film; and *(b)* PP geotextile - PE film - PP geotextile.

10.2-3 Geosynthetic Clay Liners

Geosynthetic clay liners (GCLs) are another type of composite barrier materials. A *dry* bentonite clay soil is supported between two geotextiles or on a geomembrane carrier, as illustrated in Figure 10-2. Geotextiles used above and below the *dry* clay may or may not be connected with threads or fibers, to increase the in-plane shear strength of a hydrated GCL.

Approximately 5 kg/m^2 of *dry* sodium bentonite is used in the manufacture of GCLs. The bentonite is at a moisture content of 6 to 20% in its *dry* condition. This dry bentonite hydrates and swells upon wetting, creating a very-low permeability barrier. The fully hydrated bentonite typically will have a permeability in the range of 1 to 5 x 10^{-11} m/sec.

GCLs are supplied in roll form. Widths of approximately 4.1 to 5.3 m are available. Roll lengths of 30 to 60 m are typical, although custom roll lengths may be used for large projects.

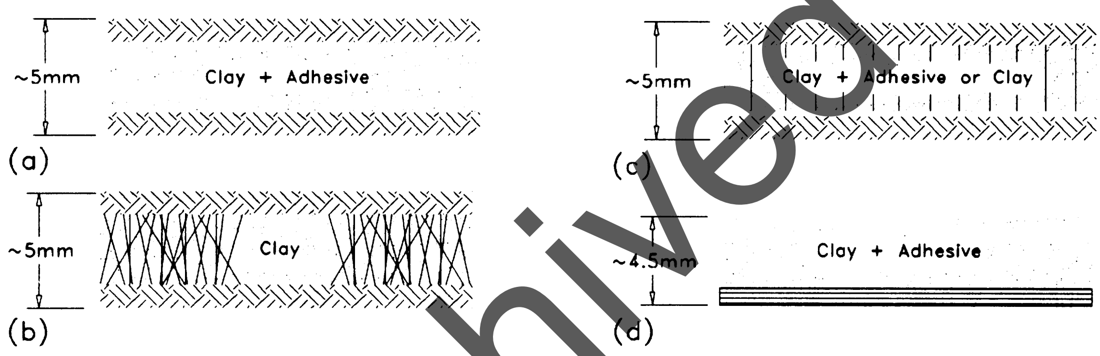

Figure 10-2 Geosynthetic clay liners: *(a)* geotextile / bentonite clay / geotextile; (*b*) stitched bonded geotextile GCL; (c) needle punched geotextile GCL; and (d) bentonite clay / PE geomembrane (after Koerner, 1994).

10.2-4 Field-Impregnated Geotextiles

Impregnated geotextiles are also used as moisture and liquid barriers. The coating treatment is applied in the field, after the geotextile is deployed and anchored. A nonwoven geotextile is used with a variety of coatings, including asphalt, rubber-bitumen, emulsified asphalt, or polymeric formulations. The coating may be proprietary. The geotextile's type and mass per unit area will be a function of the coating treatment, although use of lightweight nonwoven geotextiles, in the range of 200 to 400 g/m^2, is common. Heavier-weight, nonwoven geotextiles may be used to provide gas venting, if gas potential exists on a site.

The barrier is formed as sprayed-on liquid solidifies into a seam-free membrane. Although sprayed-on membranes are seam-free, bubbles and pinholes may form during installation and can cause performance problems. Proper preparation of the geotextile (*i.e.*, clean and dry) to be sprayed is important. These types of barriers have been used in canals, small reservoirs, and ponds for water control. Water storage applications have used air-blown asphalt coatings. (Matrecon, 1988)

Engineers also use field-impregnated geotextiles to provide moisture control in friable roadway soils. Pavement application of a barrier is called *membrane encapsulated soil layers* (MESLs).

10.3 APPLICATIONS

Geomembranes and other geosynthetic barriers are used in wide variety of applications for transportation construction and maintenance. Geomembrane transportation applications, as summarized by Koerner and Hwu (1989), are summarized below. The applications are noted as representing regular use or limited use of geomembranes and other synthetic barriers in highway works.

- Control of vertical infiltration of moisture into a subgrade of expansive soil. This minimizes the change in soil water content and subsequent volume changes. Placement of the geosynthetic barrier is illustrated in Figure 10-3. Thin-film geotextile composites or geomembranes are often used in this application.

- Control of horizontal infiltration of moisture into a subgrade of expansive soil. This minimizes change in soil water content and volume changes. Placement of the barrier is illustrated in Figure 10-4. Depth of moisture barrier is approximately 450 to 600 mm beneath estimated swell depth, or a typical total depth of 1.0 to 1.5 m. Thin-film geotextile composites or geomembranes are usually used in this application, although geomembranes and field-impregnated geotextiles are also used.

- Maintenance of water content of frost-sensitive soils with a horizontally placed barrier. This application is illustrated in Figure 10-5, as the MESL previously described. Thin-film geotextile composites or geomembranes are usually used in this application.

- Waterproofing of tunnels, as illustrated in Figure 10-6. Geomembranes (in conjunction with heavyweight nonwoven geotextiles) and GCLs are used in this application.

- Transport of water in canals lined with a geomembrane, as illustrated in Figure 10-7, or a GCL.

- Geomembranes are used for secondary containment of underground fuel storage tanks, as illustrated in Figure 10-8. GCLs and geomembranes are also used for secondary containment of above ground fuel storage tanks.

- Rest area waste water treatment lagoons may be lined with geomembranes.

- Sealing of berms for wetland mitigation.

- Storm water retention and detention ponds also may be lined with geomembranes, GCLs, or coated geotextiles.

- Geomembranes are used beneath structures as methane and radon gas barriers.

- Geomembranes are used for containment of waste, caustic soils (e.g., pyritic soils) and construction debris.

- Deicing salt and aviation deicing fluid runoff may be contained in geomembrane-lined facilities.

- Geomembranes, GCLs, and coated geotextiles may be used to waterproof walls and bridge abutments. Geomembranes may be used to prevent infiltration of corrosive deicing salt runoff into metallic MSE walls, as illustrated in Figure 10-9.

- Railroads use geosynthetic barriers to waterproof subgrades, to prevent upward groundwater movement in cuts, and to contain diesel spills in refueling areas.

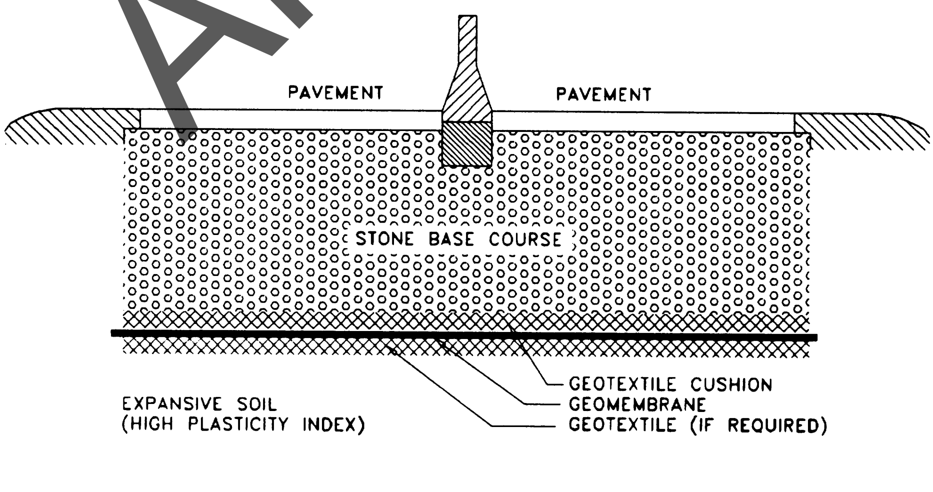

Figure 10-3 Control of expansive soils (from Koerner and Hwu, 1989).

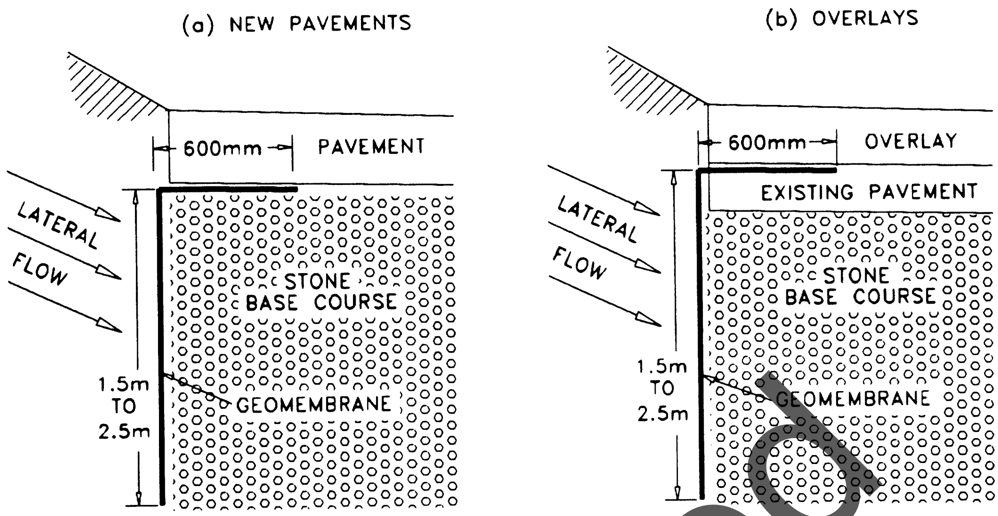

Figure 10-4 Control of horizontal infiltration of base.

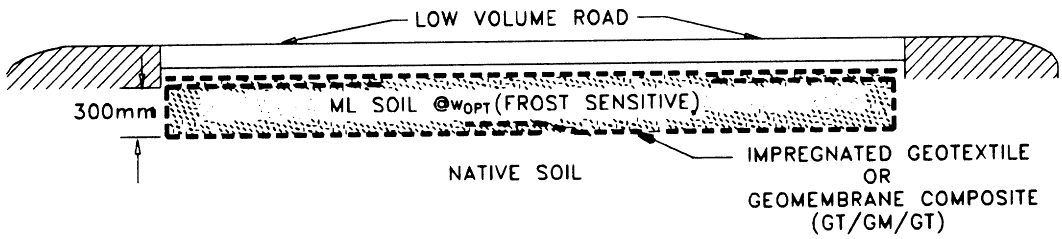

Figure 10-5 Maintenance of optimum water content (from Koerner and Hwu, 1989).

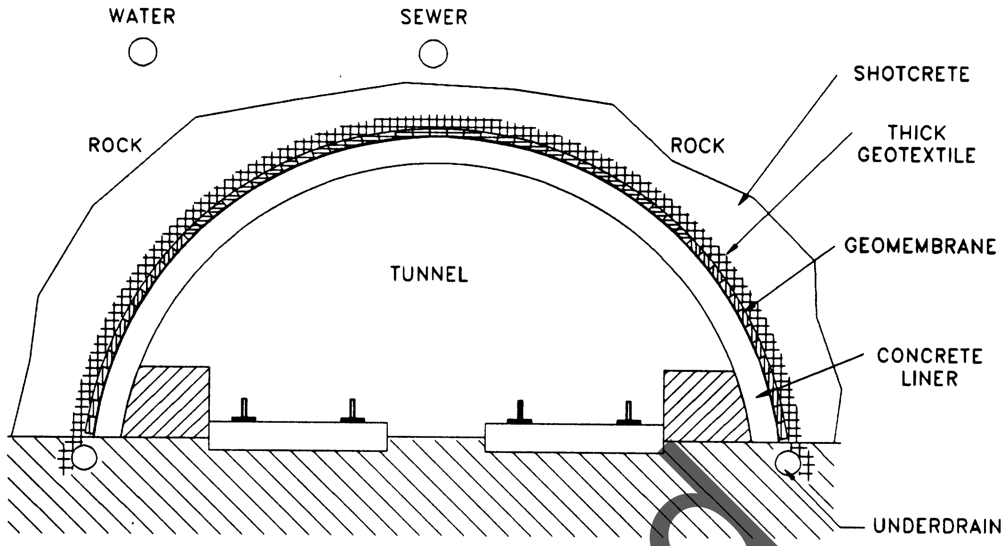

Figure 10-6 Waterproofing of tunnels (from Koerner and Hwu, 1989).

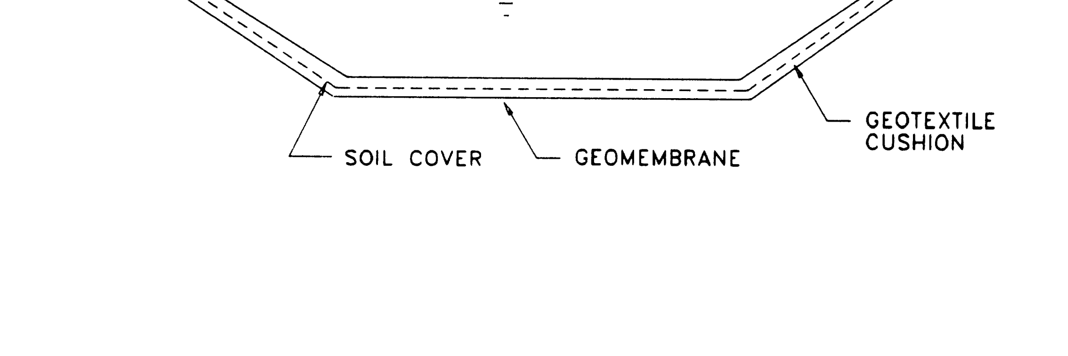

Figure 10-7 Water conveyance canals.

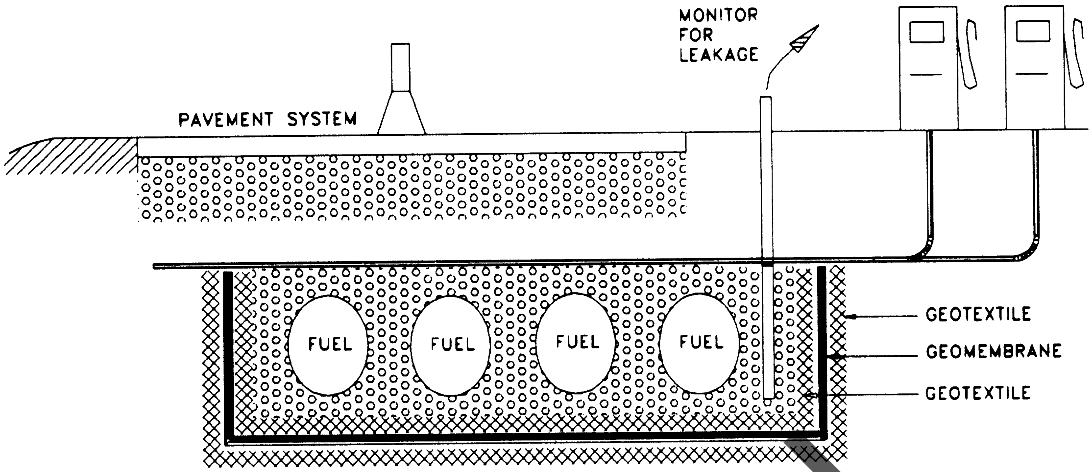

Figure 10-8 Secondary containment of underground fuel tanks (from Koerner and Hwu, 1989).

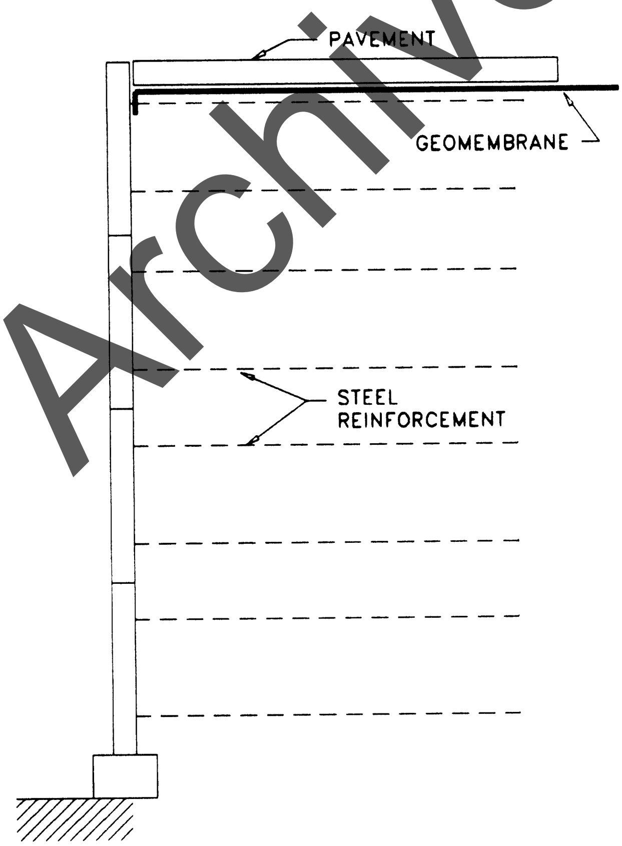

Figure 10-9 Waterproofing of walls (from Koerner and Hwu, 1989).

10.4 DESIGN CONSIDERATIONS

All geosynthetic barriers are continuous materials which are relatively impermeable as manufactured. However, to fulfill its barrier function the geosynthetic system must remain leak-proof and relatively impermeable when installed and throughout its design life. The following steps are part of the design process for geosynthetic barrier systems:

- define performance requirements;
- design for in-service conditions;
- durability design for project-specific conditions;
- design for installation, under anticipated project conditions;
- peer review (optional); and
- economic analysis.

10.4-1 Performance Requirements

The required function and performance of a geosynthetic barrier must be defined prior to design and material selection. The purpose of the barrier significantly affects the design and installation requirements. Some of the questions that should be asked regarding performance include the following.

- Is the barrier functioning as a primary or a secondary liner?
- Is a barrier less-permeable than adjacent soil required, or is an impermeable barrier system (liner(s) and drain(s)) required?
- What are the consequences of leakage?
- Can an acceptable leakage rate be defined?
- What is the anticipated life of the system, *i.e.*, is it temporary or permanent?

10.4-2 In-Service Conditions

Applications of a barrier vary, and the in-service exposure and stresses that the barrier must withstand likewise vary. Some of the questions that should be addressed regarding in-service conditions include the following.

- Will the barrier be placed within soil, or remain exposed to environmental elements throughout the design life?
- What environmental conditions (*e.g.*, temperature variations, sunlight exposure, etc.) will the geosynthetic barrier be exposed to throughout its design life?
- Will the barrier be subject to deformation-controlled (*e.g.*, due to post-construction movements caused by settlement of underlying soil) stresses ?
- Will the geosynthetic barrier be subject to downdrag forces (*e.g.*, on a side slope of a surface impoundment) or to load-controlled stresses?
- Will the barrier system be exposed to varying stress levels due to fluctuating water loads?
- Will the barrier system result in a low-friction interface that must be analyzed?

- Will the barrier trap gases and/or liquids generated beneath the liner, and require venting?
- Are performance requirements such that abrasion or puncture protection is required?

10.4-3 Durability

Many geomembranes and other geosynthetic barriers in transportation applications contain nonpolluted water. As such, geosynthetic chemical durability is not normally a concern. However, chemical resistance is a concern when liquids such as fuel or other contaminants must be contained. The chemical resistance of candidate geosynthetic barriers, as well as of their components (if applicable), must be specifically evaluated when other than nonpolluted water is to be contained. The EPA 9090 test is available for such an assessment. Available geosynthetic barriers have a wide range of chemical resistance to various elements and compounds.

Resistance to ultraviolet light must be assessed for those applications where the barrier remains exposed over its design life. Oxidation or hydrolytic degradation potential may also be assessed. Biological degradation potential should also be checked. Degradation due to vegetation growth, burrowing animals, or microorganisms may be a concern. Biological degradation of materials beneath a liner can result in gas formation that must be vented around or through the liner.

10.4-4 Installation Conditions

Installation conditions are a design consideration for all geosynthetics in all applications. Installation of geomembranes and other geosynthetic barriers is a primary design consideration. Location and installation time of year can affect barrier material selection. Environmental factors such as temperature, temperature variation, humidity, rainfall, and wind must be considered. Some geosynthetics are more sensitive to temperature than others, and moisture and wind affect field-seaming ability. Barriers constructed of field-impregnated coated geotextiles must be placed during carefully defined weather conditions.

Placement, handling, and soil covering operations can also affect geosynthetic design and selection. The panel weight and size must be compatible with project requirements and constraints. The timing of soil placement over the liner may dictate ultraviolet light resistance requirements. And the geomembrane or other geosynthetic barrier must be capable of withstanding the rigors of installation.

The subgrade material, subgrade preparation, panel deployment method, overlying soil fill type, and placement and compaction of overlying fill soil all affect the geosynthetic barrier's survivability. Recommended properties of geomembrane barriers (Koerner, 1994) are presented in Table 10-2.

TABLE 10-2
RECOMMENDED MINIMUM PROPERTIES FOR GENERAL
GEOMEMBRANE INSTALLATION SURVIVABILITY
(from Koerner, 1994)

Property and test method	Required degree of survivability			
	Low[1]	Medium[2]	High[3]	Very high[4]
Thickness, mm - ASTM D 1593	0.63	0.75	0.88	1.00
Tensile (25 mm strip), kN/m - ASTM D 882	7	8.7	10.5	12.2
Tear (Die C), N - ASTM D 1004	33	45	67	90
Puncture, N - ASTM D 3787, modified	110	130	160	180
Impact, J - ASTM D 3998, modified	9	11	15	19

NOTES:
1. *Low* refers to careful hand-placement on very uniform, well-graded subgrade with light loads of a static nature - typical of vapor barriers beneath building floor slabs.
2. *Medium* refers to hand- or machine-placement on machine-graded subgrade with medium loads - typical of canal liners.
3. *High* refers to hand- or machine-placement on machine-graded subgrade of poor texture with high loads - typical of landfill liners and covers.
4. *Very high* refers to hand- or machine-placement on machine-graded subgrade of very poor texture with high loads - typical of reservoir covers and liners for heap leach pads.

Geotextiles and other geosynthetics are often used with geomembranes to enhance the barrier's puncture resistance during installation and in-service. Geotextiles, and other geosynthetics, act as cushions and further prevent puncture of the geomembrane. The cushion can be placed below the geomembrane to resist rocks, roots, etc., in the subgrade, and/or above the geomembrane to resist puncture from subsequently placed fill or waste.

Selection of the most-effective geotextile will depend upon several factors (Richardson and Koerner, 1990), including:

- mass per unit area;
- geotextile type;
- fiber type;
- thickness under load;
- polymeric type; and
- geomembrane type and thickness.

The level of puncture protection provided by the geotextile is directly related to the mass per unit area.

10.4-5 Peer Review

A peer, or design quality assurance, review is recommended for landfill barrier systems (Rowe and Giroud, 1994). A peer review of a geosynthetic barrier structure for a transportation application may likewise be warranted, depending upon the critical nature of the structure, experience of design team, and project location and function. The goal of such a project review is to enhance the quality of the constructed project.

A peer review is recommended (Berg, 1993) for:

- projects where performance is crucial to public safety and/or the environment;
- projects that are controversial or highly visible;
- proposed designs that incorporate new materials or construction techniques;
- projects requiring state-of-the-art expertise;
- designs that lack redundancy in primary components;
- designs that have a poor performance record;
- projects with accelerated design and/or construction schedules; and
- projects with overlapping design and construction schedules.

10.4-6 Economic Considerations

Cost should be considered in design after function, performance, and installation design criteria are addressed. Material and in-place costs will obviously vary with the type of geosynthetic barrier and the quantity of barrier specified. In-place cost of geosynthetic barriers can vary from approximately $\$2.50/m^2$ to $\$16.00/m^2$. Cost of conventional compacted clay liners can vary between approximately $\$5.00/m^2$ to $\$30.00/m^2$ in-place.

10.5 INSTALLATION

A well-designed geosynthetic must be installed correctly to perform its function as a barrier. Handling and installation specifications for geomembrane and other geosynthetic barriers should, as a minimum, conform to the manufacturer's recommendations. Special project requirements should also be noted in the construction specifications and plans.

Geosynthetic barrier handling and storage requirements at the construction site should be specifically designated. Layout of the geosynthetic normally should be predetermined and documented on a roll or panel layout plan. The installer or geosynthetic supplier is normally required, by specification, to provide the layout plan.

Three areas of construction which are critical to a successful installation are:

- subgrade preparation;
- field seaming; and
- sealing around penetrations and adjacent structures.

The subgrade must provide support to the geosynthetic barrier and minimal point loadings. The subgrade must be well-compacted and devoid of large stones, sharp stones, grade stakes, etc., that could puncture the geosynthetic barrier. In general, no objects greater than 12 mm should be protruding above the prepared subgrade (Daniel and Koerner, 1993). Geotextiles are often used as cushions for geomembranes to increase puncture resistance, as previously discussed. Geotextiles and geocomposite drains are also used beneath geosynthetic barriers to vent underlying gas (*e.g.*, from decomposing organic deposits) or relieve excess hydrostatic pressure.

The method of seaming is dependent upon the chosen geosynthetic material and the project design. Overlaps, of a designated length, are typically used for thin-film geotextile composites and geosynthetic clay liners. Geomembranes are seamed with thermal methods or solvents. Temperature, time, and pressure must be specified and maintained within tolerances for thermal seaming. With solvent seams, solvent application is important, because too much solvent can weaken the geomembrane and too little solvent can result in a weak or leaky seam. Pressure, or heat, is used in conjunction with solvents. Seaming procedures for a variety of geomembranes is detailed in several waste containment manuals (Daniel and Koerner, 1993; Landreth and Carson, 1991; Eastern Research Group, Inc., 1991; Matrecon, 1988).

Construction details around penetrations and adjacent structures depend upon the chosen geosynthetic material and the project design. As such, they must be individually designed and detailed. Geosynthetic manufacturers and several waste containment manuals (Daniel and Koerner, 1993; Matrecon, 1988; Richardson and Koerner, 1988) can provide design guidance.

10.6 INSPECTION

Quality assurance (QA) and quality control (QC) are recognized as critical factors in the construction of geomembrane-lined waste containment facilities. QA and QC may or may not be as important for highway-related barrier works. The extent of QA and QC for highway barrier works should be project, and barrier product, specific.

10.6-1 Manufacture

Manufacturing quality control (MQC), normally performed by the geosynthetic manufacturer, is necessary to ensure minimum (or maximum) specified values in the manufactured product (Daniel and Koerner, 1993). Additionally, manufacturing quality assurance (MQA) programs are used to provide assurance that the geosynthetics were manufactured as specified. Quality of raw materials and of finished geosynthetic products are monitored in an MQA program. The MQA program may be conducted by the manufacturer, in a department other than manufacturing, or by an outside organization. Details on MQA and MQC for geomembrane and geosynthetic clay liners, along with other geosynthetic components, are presented in an EPA Technical Guidance Document (Daniel and Koerner, 1993).

10.6-2 Field

Construction quality assurance (CQA) and construction quality control (CQC) programs should be used for most geosynthetic barrier structure construction. CQC is normally performed by the geosynthetic installer to ensure compliance with the plans and specifications. CQA is performed by an outside organization to provide assurance to the owner and regulatory authority (as applicable) that the structure is being constructed in accordance with plans and specifications. Typically, for waste containment facilities, the CQA-performing organization is not the installer or designer, *i.e.*, it is a third-party organization. CQA may be performed by the transportation agency for highway works. Details on CQA and CQC for geomembrane and geosynthetic clay liners -- and for traditional compacted clay barriers -- are presented in an EPA Technical Guidance Document (Daniel and Koerner, 1993).

10.7 SPECIFICATION

Geosynthetic barrier specifications should contain the following components:

- statement on purpose of barrier;
- material specification for the barrier and all associated geosynthetic components, including component property requirements, product requirements, manufacturing quality control requirements, and manufacturing quality assurance;
- shipping, handling, and storage requirements;
- installation requirements;
- requirements for sealing to and around penetrations and appurtenances;
- seaming requirements, including pass/fail criteria;
- anchoring requirements; and
- statement on construction quality control and construction quality assurance.

Again, detailed information on geomembrane and GCL specifications is presented in waste management manuals (Daniel and Koerner, 1993; Matrecon, 1988).

As discussed under Installation, Section 10.5, subgrade preparation is crucial to a successful installation. Therefore, it is imperative that the accompanying subgrade preparation specification be written specifically for the geosynthetic barrier to be installed. The subgrade must be inspected and approved prior to placement of the geosynthetic barrier.

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Giroud, J.P., *Impermeability: The Myth and a Rational Approach*, Proceedings of the International Conference on Geomembranes, Vol. 1, Denver, Jun 1984, pp. 157-162.

Koerner, R.M., Designing With Geosynthetics, 3rd Edition, Prentice-Hall Inc., Englewood Cliffs, NJ, 1994, 783p.

Koerner, R.M. and Hwu, B-L , *Geomembrane Use in Transportation Systems*, Transportation Research Record 1248, Transportation Research Board, Washington, D.C., 1989, pp. 34-44.

Landreth, R.E. and Carson, D.A., Inspection Techniques for the Fabrication of Geomembrane Field Seams, U.S. EPA Technical Guidance Document, EPA/530/SW-91/051, Cincinnati, OH, 1991, 174 p.

Matrecon, Inc., Lining of Waste Containment and Other Impoundment Facilities, U.S. EPA Technical Guidance Document, EPA/600/2-88/052, Cincinnati, OH, 1988, 1026 p.

Richardson, G.N. and Koerner, R.M., Editors, A Design Primer: Geotextiles and Related Materials, Industrial Fabrics Association International, St. Paul, MN, 1990, 166 p.

Richardson, G.N. and Koerner, R.M., Geosynthetic Design Guidance for Hazardous Waste Landfill Cells and Surface Impoundments, EPA Contract No. 68-03-3338, U.S. Environmental Protection Agency, Cincinnati, OH, 1988.

Rowe, R.K. and Giroud, J.P., *Quality Assurance of Barrier Systems for Landfills*, IGS News, International Geosynthetics Society, Vol. 10, No. 1, Mar 1994, pp. 6-8.

Staff, C.E., *The Foundation and Growth of the Geomembrane Industry in the United States*, Proceedings of International Conference on Geomembranes, Denver, Jun 1984, pp. 5-8.

APPENDICES

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Appendix A
GEOSYNTHETIC LITERATURE

A-1 **1988 List** (Holtz, R.D. and Paulson, J.N., *Geotechnical News*, Vol. 6, No. 1, March 1988, pp. 13-15)

Introduction

In only a very few years, geosynthetics have joined the list of possible solutions to a number of important civil engineering problems. Two important and increasingly critical examples come to mind immediately; the control of hazardous waste containment and soil reinforcement. In many cases, the use of a geosynthetic can significantly increase the structure's safety factor, improve performance, and reduce costs in comparison with conventional solutions.

The range of applications of geotextiles, geogrids, and related products is enormous. They are used in:

- filtration, drainage, and in erosion protection and control systems.
- stabilization of roadways and railroads on soft subgrades.
- reinforcement of retaining walls, earth and waste slopes, and embankments.

Geomembranes are primarily used as:

- pond and canal liners
- barriers in hazardous and other waste containment systems
- rehabilitation of old earth fill and concrete dams.
- environmental control in highways and other civil engineering construction.

Geotextiles, geogrids, and geomembranes can also be used together in a *Geocomposite* system to provide multiple functions. Common applications of geocomposites are in hazardous waste containment systems and as prefabricated drainage layers.

Even with the rapid recent growth in the use of these materials, it has been our experience that civil engineers often have difficulty obtaining what they consider to be reliable and impartial information about design, specifications, and construction with geosynthetics. This perceived lack of information is primarily a communications problem, because a relatively large body of information on geosynthetics has been published in books, journals, and conference proceedings.

This article presents some sources of information which are readily available to civil engineers contemplating design and construction with geosynthetics. Included are:

- books
- journals and other periodicals
- conference proceedings
- reports

In some casts, addresses where publications may be obtained are also given.

The field of geosynthetics is developing so rapidly that it can be difficult for the non-specialist to remain current. To assist in this regard, we also list organizations and technical committees concerned with geosynthetics. Watch for announcements of their meetings, symposia, and conferences, because that is where the latest information, research findings, and case histories are presented and discussed.

A. General References, Books and Manual

Bell, J.R., Hicks, R.G., et al, (1980 and 1982) "*Evaluation of Test Methods and Use Criteria for Geotechnical Fabrics in Highway Applications*," Oregon State University, Corvallis, Interim Report No. FHWA/RD-80/021, Final Report No. FHWA/RD-82.

Christopher, B.R. and Holtz, R.D. (1985) *Geotextile Engineering Manual*, STS Consultants Ltd., Northbrook, Illinois; Federal Highway Administration, Report FHWA-TS-86/203, 1044 p. (to be published by I.F.A.I.).

Giroud, J.P. (1984) *Geotextiles and Geomembranes, Definitions, Properties and Design*, I.F.A.I., Suite 450, 345 Cedar Street, St. Paul, MN 55101, 325 p.

Jones, C.J.F.P, (1985) *Earth Reinforcement and Soil Structures*, Butterworths, London, 183 p.

Kays, W.B. (1977) *Construction of Linings for Reservoirs, Tanks and Pollution Control Facilities*, John Wiley and Sons, 1977.

Koerner, R.M. and Welsh, J.P. (1980) *Construction and Geotechnical Engineering Using Synthetic Fabrics*, John Wiley and Sons, 267 p.

Koerner, R.M. (1986) *Designing with Geosynthetics*, Prentice-Hall, Inc., 424 p.

Rankilor, P.R. (1981) *Membranes in Ground Engineering*, John Wiley and Sons, 377 p.

Scott, J.D. and Richards, E.A. (1985) *Geotextile and Geomembrane International Information Source*, BiTech Publications, 902-1030 West Georgia Street, Vancouver, B.C., Canada, V6E 2Y3.

Steward, J., Williamson, R. and Mohney, J., (1977) "Guidelines for Use of Fabrics in Construction and Maintenance of Low-Volume Roads," USDA, Forest Service, Portland, Oregon. Also published by FHWA as Report No. FHWA-TS-78-205.

Veldhuijzen van Zanten, R. Ed. (1986) *Geotextiles and Geomembranes in Civil Engineering*, John Wiley, 658 p.

B. Periodicals and Journals

Specifically devoted to geosynthetics:

Geotechnical Fabrics Report, published by Industrial Fabrics Association International (IFAI), Suite 450, 345 Cedar Street, St. Paul, MN 55101, (612-22-2508). First issue Summer, 1983; now six issues/year; $30 (complimentary with membership in the North American Geosynthetics Society).

Geotextiles and Geomembranes, published by Elsevier Applied Science Publishers, Crown House, Linton Road, Barking, Essex IGII 8JU, England. First volume published in 1984; now four issues/year; $128 ($77 for members of the International Geotextile Society).

Useful technical articles on geosynthetics also occasionally appear in the standard geotechnical journals:

- *Journal of Geotechnical Engineering (ASCE)*
- *Geotechnical Testing Journal - (ASTM)*
- *Canadian Geotechnical Journal (CGS)*
- *Transportation Research Record (TRB)*

C. Proceedings of Conferences, Symposia, Etc.

1. International:

International Conference on the Use of Fabrics in Geotechnics (1977), Paris. Proceedings available from Ecole Nationale des Ponts et Chausees, 28 rue de Saints-Peres, Paris, France (2 Vols.).

Second International Conference on Geotextiles (1982), Las Vegas, NV. Proceedings available from IFAI (4 Vols.).

Third International Conference on Geotextiles (1986), Vienna. Proceedings available from IFAI (5 Vols.).

2. Canadian:

Proceedings of the 1st Canadian Symposium on Geotextiles (1980), Canadian Geotechnical Society, Calgary, Alberta.

Proceedings of the Second Canadian Symposium of Geotextiles and Geomembranes (1985), Edmonton, Alberta.

Seminar on the Use of Synthetic Fabrics in Civil Engineering (1981), Consulting Engineers of Ontario, Toronto, Ontario.

3. Other Geotextile Publications:

The Use of Geotextiles for Soil Improvement (1980) Preprint 80-177 ASCE National Convention, Portland, OR.

"Fabrics," Special Supplement to *Civil Engineering* (England), March, 1981.

"Engineering Fabrics in Transportation Construction," (1983) *Transportation Research Record* 916.

Symposium of Polymer Grid Reinforcement in Civil Engineering (1984) London, Available from Tensar Corp., Atlanta, GA.

"Geotextiles as Filters and Transitions in Fill Dams" (1986) *Bulletin 55* International Commission on Large Dams; available from USCOLD, Denver, CO.

Proceedings of the *Conference to Geosynthetics '87*, (1987) New Orleans, Louisiana (2 Vols.), sponsored by IFAI, NAGS, and IGS. Proceedings available from IFAI (NOTE: Also contains papers on geomembranes.)

Fluet, J.E. (editor) (1987) *Geotextile Testing and the Design Engineer* ASTM Special Technical Publication 952.

4. Geomembranes:

National Conference on Management of Uncontrolled Hazardous Waste Substances (1981) Hazardous Materials Control Research Institute Silver Springs, MD.

Management of Uncontained Hazardous Waste Sites (1983), Hazardous Materials Control Research Institute, Silver Springs, MD.

International Conference on Geomembranes (1984) Denver. Proceedings available from IFAI.

Johnson, A.I., Frobel, R.K., Cavaiti, N.J., and Pettersson, C.B. (editors) (1985) *Hydraulic Barriers in Soil and Rock*, ASTM, STP 874, 332 p.

Vanzyl, D.J.A., Abt, S.R., Nelson, J.D. and Shepard, T.A. (editors) (1987) *Geotechnical and Geohydrological Aspects of Waste Management*, Lewis Publishers, Inc., Chelsea, Michigan, 312 p.

D. Manufacturer's Brochures, Product Specifications, and Design Manuals

A number of manufactures have brochures describing typical applications, and listing the general and index properties of their products. Some have also produced detailed design manuals, often written by reputable consulting engineers and professors. In some instances, they have technical assistance personnel on staff. This information and assistance is usually free upon request and should be considered along with the other information given above.

E. Other Information Sources

A number of societies and committees have been formed and are good sources of current information to keep you up to date. Groups interested in geosynthetics (chairpersons in parenthesis):

ASCE: Geotechnical Engineering Division Committee on Placement and Improvement of Soils (L.R. Anderson: 801-750-2775). For publication information, contact ASCE (212-705-7538).

ASTM: Committee on Geotextiles, Geomembranes, and Related Products (B.R. Christopher: 312-272-6520). For publication information, contact ASTM (215-297-5400)

TRB: Committee A2K07 on Geosynthetics (V.C., McGuffey: 518-457-4712). For publication information, contact TRB (202-334-3218).

ISSMFE: Committee on Geotextiles (J.P.Giroud: 305-736-5400).

International Geotextile Society (IGS) (J.P. Giroud: 305-736-5400).

North American Geosynthetic Society (NAGS) (J.E. Fluet: 305-736-5400).

A-2 1992 List (Cazzuffi, D. and Anzani, A., *IGS News*, Vol. 8, No. 1, March, 1992)

The list of publications given in this document has been compiled by the Members of the IGS Education Committee under the guidance of D. Cazzuffi and Anna Anzani. The list does not purport to be complete but is offered as a starting point for those readers interested in acquiring recognized high-quality publications on geotextiles, geomembranes and related products.

The list of reference documents include conference proceedings, textbooks and magazines and have been grouped into the following categories:

- General Topics
- Material Characteristics and Testing
- Reinforcement Applications
- Dams
- Bank Protection
- Waste Containment Applications.

In each category the reference documents have been listed in chronological order (except for General Topics, where Conference Proceedings have been grouped first, followed by textbooks and magazines).

This list has been reviewed by all Members of the IGS Education Committee.

General Topics

Proceedings of the International Conference on the Use of Fabrics in Geotechnics - First International Conference on Geotextiles (1977) - Paris (3 volumes, 532 pages). Order from: ENPC, Service Formation Continue, 28 Rue des Saints Peres, 75006 Paris, France.
(Price: US $ 50 plus postage)

Proceedings of the Second International Conference on Geotextiles (1982) - Las Vegas (4 volumes, 1024 pages). Order from: IFAI, 345 Cedar St., Suite 800, St. Paul, MN 55101, USA.
(Price: US $72 plus postage)

Proceedings of the Third International Conference on Geotextiles (1986) - Wien (5 volumes, 1550 pages).
Order from; IFAI, see address above (for North America).
(Price: US $ 128 plus postage)
Balkema, P.E. Box 1875, NL - 3000 BR Rotterdam, The Netherlands (for the rest of the world).
(Price: 300 dFl plus postage)

Proceedings of the Fourth International Conference on Geotextiles, Geomembranes, and Related Products (1990) - The Hague (2 volumes, 884 pages, plus a third volume in print).
Order from: Balkema, see address above.
(Price: 290 dFl plus postage)
Balkema, Old Post Road, Brookfield, VT 06036, USA (for North America). (Price: US $160 plus postage).

Proceedings of the International Conference on Geomembranes (1984) - Denver, (2 volumes, 511 pages). Order from: IFAI - see address above.
(Price: US $40 plus postage)

Proceedings of Geosynthetics '87 (1987) - New Orleans (2 volumes, 639 pages).
Order from: IFAI, see address above. (Price: US $50 plus postage)

Proceedings of Geosynthetics '89 (1989) - San Diego (2 volumes, 600 pages). Order from: IFAI, see address above. (Price: US $55 plus postage)

Proceedings of Geosynthetics '91 (1991) - Atlanta (2 volumes, 864 pages). Order from: IFAI, see address above. (Price: US $55 plus postage)

Rankilor, P.R. (1981), Membranes in Ground Engineering, John Wiley and Sons Ltd., Chichester, UK (377 pages). Order from: J. Wiley, Baffins Lane, Chichester, West Sussex, PO191UD, UK.

Giroud, J-P. (1985), Geotextiles and Geomembranes, Definitions, Properties and Design, IFAI, St. Paul, MN, USA (404 pages). Order from: IFAI, see address above. (Price: US $49 plus postage).

van Zanten, R.V. - Editor (1986), Geotextiles and Geomembranes in Civil Engineering, John Wiley & Sons Ltd., Chichester, U.K. Order from: John Wiley, see address above.

John , N.W.M. (1987), Geotextiles. Blackie and son Ltd., Glasgow and London, UK.
Order from: Blackie and son Ltd., 7 Leicester Place, London W2CH 7BP, UK.

Koerner, R.M. (1990), Designing with Geosynthetics, Prentice Hall, Englewood Cliffs, NJ, USA (652 pages).
Order from: IFAI, see address above. (Price: US $70 plus postage).

Venkatappa, R. G. and Raju, G.V.S.S. - Editors (1990). Engineering with Geosynthetics, Tata McGraw-Hill, New Delhi, (316 pages). Order from: Tata McGraw-Hill Publishing Company Ltd., 4/12 Asaf All Road, New Delhi - 110 002 (Price: Rs. 215/-)

Haussmann, M.R. (1991), Engineering Principles of Ground Modifications, McGraw-Hill, New York, USA (632 pages).
Order from: McGraw-Hill Book Company, International Group, 1221 Avenue of the Americans, New York, NY 10020, USA.

Geotextiles and Geomembranes (Editor T.S. Ingold), an official journal of the IGS published by Elsevier in six issues per year. Order from: Elsevier Science Publisher Ltd., Crown House, Linton Road, Barking, Essex IG1 8JU, UK. (Price £ 160/year) Elsevier Science Publishing Co., Inc., Journal Information Center, 655 Avenue of the Americas, New York, NY 10010, USA (for North American) (Price: US $296/year)

Material Characteristics and Testing

Fluet, J. - Editor (1987), Geotextile Testing and the Design Engineer, ASTM, Philadelphia, USA (192 pages). Order from: ASTM European Office, 27/29 Wilbury Way, Hitchin, Herts SG4 OSX, UK (for Europe). (Price: £ 25 plus postage) ASTM, 1915 Race Street, Philadelphia, PA 19103, USA.

Rilem (1988), Durability of Geotextiles, Chapman and Hall, London (230 pages). Order from: E & FN Spon. Marketing Dept., 2-6 Boundary Row, London SE18HN, UK. (Price: £ 29 plus postage).

Peggs, I.D. - Editor (1990), Geosynthetics, Microstructure and Performance, ASTM, Philadelphia, USA (170 pages) . Order from: ASTM European Office, see address above. (Price: $26 plus postage).

Rollin, A. and Rigo, J.M. - Editors (1991), Geomembranes, Identification and Performance Testing, Chapman and Hall, London (376 pages). Order from: E & FN Spon, see address above. (Price: £ 45 plus postage).

Reinforcement Applications

Jones, J.F.P. (1984), Earth Reinforcement and Soil Structures, Butterworths, London (184 pages). Order from: Butterworths, Borough Green, Sevenoaks, Kent TN158PH, UK (Price: £ 30 plus postage).

ASCE (1987), Soil Improvement - A Ten Year Up-To-Date, ASCE Geotechnical Special Publication No. 12. Order from: ASCE, 345 East 47th Street, New York, NY 10017, USA.

Jarrett, P.M. and McGown, A. - Editors (1987), The Application of Polymeric Reinforcements in Soil Retaining Structures. Kluwer Academic Publisher. Dordrecht, The Netherlands (638 pages). Order from: Kluwer Academic Publisher Group, P.O. Box 322, 3300 AH Dordrecht, NL (Price: 275 hFl plus postage). Kluwer Academic Publisher, 101 Philip Drive, Norwell, MA 02061, USA (Price: US $149 plus postage).

Mitchell, J.K, and Villet, W.C.B., (1987), Reinforcement of Earth Slopes and Embankments, National Cooperative Highway Research Program Report #290, Transportation Research Board.

Proceedings of the International Geotechnical Symposium: Theory and Practice of Earth Reinforcements, (1988) - Fukuoka (618 pages). Order from: Balkema, P.O. Box 1675, NL - 3000 BR Rotterdam, The Netherlands. (Price 120 hFl plus postage).
Balkema, Old Post Road, Brookfield, VT 06036, USA (for North America). (Price: US $59)

Rigo, J.M. and Degeimbre, R. - Editors (1989), Reflective Cracking in Pavements, Assessment and Control, Liege University. Order from: Universite de Liege, Inst. du Ganie Civil, Qual Banning, 6, B-4000 Ljege.

Shercliffe, D.A. - Editor (1990), Reinforced Embankments - Theory and Practice, Thomas Telford, London (177 pages). Order from: Thomas Telford, Ltd., Thomas Telford House, 1 Heron Quay, London E144JD, UK.

McGown, A., Yoe, K.C. and Andrawes, K.Z. - Editors (1991), Performance of Reinforced Soil Structures, Thomas Telford, London (485 pages). Order from: Thomas Telford Ltd., see address above.

Dams

ICOLD (1986), Geotextiles as Filters and Transitions in Fill Dams, Paris (130 pages). Order from: Commission Internationale des Grands Barrages, 151, bd Haussmann, 75008 Paris, France. (Price: Ff. 100 plus postage).

ICOLD (1991), Watertight Geomembranes for Dams, State of the Art, Paris (140 pages). Order from: CIGB, see address above. (Price: Ff. 180 plus postage).

Bank Protection

Flexible Armoured Revetments Incorporating Geotextiles (1984), T. Telford, London (400 pages). Order from: Thomas Telford Ltd., see address above. (Price: £ 29.94 plus postage).

PIANC (1987), Guidelines for the Design and Construction of Flexible Revetments Incorporating Geotextiles for Inland Waterways, PIANC, Bruxelles (156 pages). Order from: Thomas Telford Ltd., see address above. (Price: £ 20 plus postage).

Memphill, R. W. and Bramley, M.E. (1989) Protection of River and Canal Banks, CIRIA - Butterworths (200 pages). Order from: Butterworths, see address above. (Price: £ 45 plus postage).

Waste Containment Applications

Proceedings of the Second International Landfill Symposium. Sardinia 89, (1989) - Porto Conte (Alghero) (2 volumes, 1,220 pages). Order from: CIPA, Via Palladio 25, I-20235, Milano. (Price: Lit 250000 plus postage)

Koerner, R.M. - Editor (1990), Geosynthetics Testings for Waste Containment Applications, ASTM, Philadelphia (386 pages). Order from: ASTM European Office, see address above (Price: £ 33 plus postage).

ISSFME - ETC (1991), Technics of Landfills and Contaminated Land. Technical Recommendations "GLC", ed. by the German Geotechnical Society for the ISSMFE, Ernst, Berlin (76 pages). Order from: Hans L. Jessberger, Ruhr University, Bochum, P.O. Box 102148, D-4630, Bochum 1, Germany.

Proceedings of the Third International Landfill Symposium, Sardinia 91 (1991) - Cagliari (2 volumes, 1,816 pages). Order from: CISA, Via Marengo 34, I-09123, Cagliari (Price: Lit 400000 plus postage).

Appendix B
GEOSYNTHETIC TERMS

apparent opening size (AOS, O_{95}) - a property which indicates the approximate largest particle that would effectively pass through a geotextile

blinding - condition whereby soil particles block the surface openings of a geotextile, thereby reducing hydraulic conductivity

California Bearing Ratio (CBR) - the ratio of (1) the force per unit area required to penetrate a soil mass with a 3-square-inch circular piston (approximately 2-inch diameter) at the rate of 0.05 inches/minute to (2) the force per unit area required for corresponding penetration of a standard material

clogging - condition where soil particles move into and are retained in the openings of a geotextile, thereby reducing hydraulic conductivity

cross-machine direction - the direction in the plane of the geosynthetic perpendicular to the direction of manufacture

filtration - the process of retaining soils while allowing the passage of water (fluid)

geocell - a three-dimensional comb-like structure, to be filled with soil or concrete

geocomposite - a geosynthetic material manufactured of two or more materials

geogrid - a geosynthetic formed by a regular network of tensile elements and apertures, typically used for reinforcement applications

geomembrane - an essentially impermeable geosynthetic, typically used to control fluid migration

geonet - a geosynthetic consisting of integrally connected parallel sets of ribs overlying similar sets of ribs, for planar drainage of liquids or gases

geosynthetic - a planar product manufactured from polymeric material used with soil, aggregate, or other geotechnical engineering materials

geotextile - a permeable geosynthetic comprised solely of textiles

index test - a test procedure which may contain a known bias but which may be used to establish an order for a set of specimens with respect to the property of interest

machine direction - the direction in the plane of the geosynthetic parallel to the direction of manufacture

permeability - the rate of flow of a liquid under a differential pressure through a material

permittivity - the volumetric flow rate of water per unit cross sectional area per unit head under laminar flow conditions, in the normal direction through a geotextile

Bibliography

Standard Terminology for Geosynthetics, ASTM D 4439, American Society for Testing and Materials, Philadelphia, PA, 1994, 3 p.

Frobel, R.K., Geosynthetics Terminology - An Interdisciplinary Treatise, Industrial Fabrics Association International, St. Paul, 1987, 126 p.

Appendix C
NOTATION AND ACRONYMS

C-1 NOTATION

a	=	radius of tire contact area
A	=	area
AOS	=	apparent opening size
b	=	a dimension; horizontal length of embankment slope
B	=	a coefficient; width of geosynthetic or embankment
c	=	undrained shear strength ("cohesion") in terms of total stresses
c'	=	effective stress strength parameters
c_a	=	soil-geosynthetic adhesion
c_v	=	coefficient of consolidation
C_c	=	compressive index
C_r	=	recompression index
C_u	=	uniformity coefficient, D60/D10
CBR	=	California Bearing Ratio
d	=	depth
D	=	grain size (subscript indicates percent smaller than); depth of embankment; thickness of soft layers
e	=	eccentricity
F*	=	the pullout resistance (or friction-bearing interaction) factor
FS	=	factor of safety
FS_{cr}	=	partial factor for creep deformation, ratio of T_{ult} to creep limiting strength,
FS_{ID}	=	partial factor of safety for installation damage
FS_{CD}	=	partial factor of safety for chemical degradation
FS_{BD}	=	partial factor of safety for biological degradation
FS_{JNT}	=	partial factor of safety for joints, seams, and connections
g	=	acceleration due to gravity
G_L	=	lower strength geogrid
G_H	=	higher strength geogrid
GVW	=	gross vehicle weight
H	=	head difference (gradient ratio test); embankment, slope or wall height
i	=	hydraulic gradient
k	=	coefficient of permeability
K	=	stress ratio; force coefficient
K_A	=	active earth coefficient of the retained backfill
L	=	length; length of reinforcement; length of failure arc
L_E	=	embedment length to resist pullout
M	=	moment
n	=	porosity
N	=	number of layers
N_C	=	bearing capacity factor for cohesive soils
O	=	opening size; subscript indicates percent smaller than
P_a	=	active earth pressure

P_b = resultant active earth pressure due to the retained backfill
PI = plasticity index
P_q = resultant active earth pressure due to the uniform surcharge
P_Q = resultant of live load
q = flow rate; surcharge load
q_a = allowable bearing capacity
q_{ult} = ultimate bearing capacity
Q_L = live load
R = radius of critical failure circle
R_v = resisting force (Meyerhof's approach)
S = vertical spacing between horizontal geogrid layer
SF = safety factor
t = thickness of geogrid
T = tensile strength of the geosynthetic
T_a = allowable tensile strength of the geosynthetic
T_d = design tensile strength of the geosynthetic (usually at a given strain)
T_{ult} = ultimate tensile strength of a geosynthetic
T_l = creep limit tensile strength of a geosynthetic
v = vertical
V_q = vertical force due to surcharge
w = water content
W = vertical force due to the weight of the fill
x = a dimension or coordinate
y = a dimension or coordinate
α = peak horizontal acceleration for seismic loading
β = slope of soil surface; angle of reinforcement force
γ = unit weight
Δ = change in some parameter or quantity
ϵ = strain
Θ = inclination of the wall face
μ = friction coefficient along the sliding plane, which depends on the location plane, *i.e.*, tan ϕ_r or tan ϕ_f
μ^* = the pullout resistance of shearing friction between soil and geogrid
ψ = permittivity
θ = transmissivity; an angle; angle of failure plane
σ_h = horizontal stress
σ_o = overburden stress
σ_p' = preconsolidation stress
σ_v = vertical stress
ϕ = angle of internal friction
ϕ' = effective angle of internal friction
τ = shear resistance

C-2 ACRONYMS

AASHTO	American Association of State Highway and Transportation Officials
CPE	chlorinated polyethylene
CSPE	chlorosulfonated polyethylene
CQA	construction quality assurance
CQC	construction quality control
EIA	ethylene interpolymer alloy
EPA	U.S. Environmental Protection Agency
FHWA	U.S. Department of Transportation, Federal Highway Administration
HDPE	high-density polyethylene
GCL	geosynthetic clay liner
MCU	modular concrete units
MSE	mechanically stabilized earth
MQA	manufacturing quality assurance
MQC	manufacturing quality control
MESL	membrane encapsulated soil layer
PP	polypropylene
PVC	polyvinyl chloride
QA	quality assurance
QC	quality control
SRW	segmental retaining wall (unit) - see MCU
VLDPE	very low-density polyethylene
USFS	U.S. Department of Agriculture, Forest Service

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Appendix D
AASHTO M288 SPECIFICATION

(Taken from *Standard Specifications for Transportation Materials and Methods of Sampling and Testing*. Copyright 1997. By the American Association of State Highway and Transportation Officials. Reproduced with permission.)

Standard Specification
for

Geotextile Specification for Highway Applications

AASHTO DESIGNATION: M 288-96

1. SCOPE

1.1 This is a materials specification covering geotextile fabrics for use in subsurface drainage; separation; stabilization; erosion control; temporary silt fence; and paving fabrics. This is a material purchasing specification and design review of use is recommended.

1.2 This is not a construction or design specification. This specification is based on geotextile survivability rom installation stresses. Refer to Appendix A of this specification for geotextile construction guidelines.

2. REFERENCED DOCUMENTS

2.1 *AASHTO Standards*

T 88 Particle Size Analysis of Soils◆
T 90 Determining the Plastic Limit and Plasticity Index of Soils
T 99 The Moisture-Density Relations of Soils Using a 5.5 lb. (2.5 kg) Rammer and a 12 in. (305 mm) Drop

2.2 *ASTM Standards:*[1]

D 123 Standard Terminology Relating to Textiles
D 276 Test Method for Identification of Fibers in Textiles
D 378 Test Method for Hydraulic Bursting Strength of Knitted Goods and Nonwoven Fabrics — Diaphragm Bursting Strength Tester Method
D 4354 Practice for Sampling of Geosynthetics for Testing
D 4355 Test Method for Deterioration of Geotextiles from Exposure to Ultraviolet Light and Water (Xenon-Arc Type Apparatus)
D 4439 Terminology for Geosynthetics
D 4491 Test Methods for Water Permeability of Geotextiles by Permittivity
D 4533 Test Method for Trapezoid Tearing Strength of Geotextiles
D 4632 Test Method for Grab Breaking Load and Elongation of Geotextiles
D 4751 Test Method for Determining Apparent Opening Size of a Geotextile
D 4759 Practice for Determining the Specification Conformance of Geosynthetics
D 4833 Test Method for Index Puncture Resistance of Geotextiles, Geomembranes, and Related Products
D 4873 Guide for Identification, Storage, and Handling of Geotextiles
D 5141 Test Method to Determine Filtering Efficiency and Flow Rate for Silt Fence Applications Using Site Specific Soils

[1] Available from ASTM, 1916 Race Street, Philadelphia, PA 19103-1187.

2.3 Texas Department of Transportation, Manual of Testing Procedures[2]

TEX 616-J Asphalt Retention and Potential Change of Area

3. PHYSICAL REQUIREMENTS

3.1 Fibers used in the manufacture of geotextiles, and the threads used in joining geotextiles by sewing, shall consist of long-chain synthetic polymers, composed of at least 95 percent by weight of polyolefins or polyesters. They shall be formed into a stable network such that the filaments or yarns retain their dimensional stability relative to each other, including selvages.

3.2 Geotextiles used for subsurface drainage, separation, stabilization, an permanent erosion control applications shall conform to the physical requirements of Section 7. Geotextiles used for temporary silt fence shall conform to the physical requirements of Section 8 and geotextiles used as paving fabrics shall conform to the physical requirements of Section 9.

3.3 All property values, with the exception of apparent opening size (AOS), in these specifications represent minimum average roll values (MARV) in the weakest principle direction (i.e., average test results of any roll in a lot sampled for conformance or quality assurance testing shall meet or exceed the minimum values provided herein). Values for AOS represent maximum average roll values.

4. CERTIFICATION

4.1 The contractor shall provide to the Engineer, a certificate stating the name of the manufacturer, product name, style number, chemical composition of the filaments or yarns and other pertinent information to fully describe the geotextile.

4.2 The Manufacturer is responsible for establishing and maintaining a quality control program to assure compliance with the requirements of the specification. Documentation describing the quality control program shall be made available upon request.

4.3 The Manufacturer's certificate shall state that the furnished geotextile meets MARV requirements of the specification as evaluated under the Manufacturer's quality control program. The certificate shall be attested to by a person having legal authority to bind the Manufacturer.

4.4 Either mislabeling or misrepresentation of materials shall be reason to reject those geotextile products.

5. SAMPLING, TESTING, AND ACCEPTANCE

5.1 Geotextiles shall be subject to sampling and testing to verify conformance with this specification. Sampling for testing shall be in accordance with ASTM D 4354. Acceptance shall be based on testing of either conformance samples obtained using Procedure A of ASTM D 4354, or based on manufacturer's certifications and testing of quality assurance samples

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[2] Available from Materials and Tests Division of the Texas Department of Transportation, 125 E. 11th St., Austin, Texas 78701-2483.

obtained using Procedure B of ASTM D 4354. A lot size for conformance or quality assurance sampling shall be considered to be the shipment quantity of the given product or a truckload of the given product, whichever is smaller.

5.2 Testing shall be performed in accordance with the methods referenced in this specification for the indicated application. The number of specimens to test per sample is specified by each test method. Geotextile product acceptance shall be based on ASTM D 4759. Product acceptance is determined by comparing the average test results of all specimens within a given sample to the specification MARV. Refer to ASTM D 4759 for more detail regarding geotextile acceptance procedures.

6 SHIPMENT AND STORAGE

6.1 Geotextiles labeling, shipment, and storage shall follow ASTM D 4873. Product labels shall clearly show the manufacturer or supplier name, style name, and roll number. Each shipping document shall include a notation certifying that the material is in accordance with the manufacturer's certificate.

6.2 Each geotextile roll shall be wrapped with a material that will protect the geotextile from damage due to shipment, water, sunlight, and contaminants. The protective wrapping shall be maintained during periods of shipment an storage.

6.3 During storage, geotextile rolls shall be elevated off the ground and adequately covered to protect them from the following: site construction damage, precipitation, extended ultraviolet radiation including sunlight, chemicals that are strong acids or strong bases, flames including welding sparks, temperatures in excess of 71°C (160°F), and any other environmental condition that may damage the physical property values of the geotextile.

7. GEOTEXTILE PROPERTY REQUIREMENTS FOR SUBSURFACE DRAINAGE, SEPARATION, STABILIZATION, AND PERMANENT EROSION CONTROL

7.1 General Requirements

7.1.1 Table 1 provides strength properties for three geotextile classes. The geotextile shall conform to the properties of Table 1 based on the geotextile class required in Table 2, 3, 4, or 5 for the indicated application.

7.1.2 All numeric values in Table 1 represent MARV in the weaker principal direction. The geotextile properties required for each class are dependent upon geotextile elongation. When sewn seams are required, the seam strength, as measured in accordance with ASTM D 4632, shall be equal to or greater than 90 percent of the specified grab strength.

7.2 Subsurface Drainage Requirements

7.2.1 Description. This specification is applicable to placing a geotextile against a soil to allow for long-term passage of water into a subsurface drain system retaining the in-situ soil. The primary function of the geotextile in subsurface drainage applications is filtration. Geotextile filtration properties are a function of the in-situ soil gradation, plasticity, and hydraulic conditions.

7.2.2 Geotextile Requirements. The geotextile shall meet the requirements of Table 2. Woven slit film geotextiles (i.e., geotextiles made from yarns of a flat, tape-like character) will not be allowed. All numeric values in Table 2, except AOS represent MARV in the weaker principal direction. Values of AOS represent maximum average roll values.

7.2.3 The property values in Table 2 represent default values which provide sufficient geotextile survivability under most construction conditions. Note 2 of Table 2 provides for a reduction in the minimum property requirements when sufficient survivability information is available. The Engineer may also specify properties different from those listed in Table 2 based on engineering design experience.

7.3 Separation Requirements

7.3.1 Description. This specification is applicable to the use of a geotextile to prevent mixing of a subgrade soil and an aggregate cover material (subbase, base, select embankment, etc.). This specifications may also apply to situations other than beneath pavements where separation of two dissimilar materials is required but where water seepage through the geotextile is not a critical function.

7.3.2 The separation application is appropriate for pavement structures constructed over soils with a California Bearing Ratio equal to or greater than 3 (CBR $\geq$ 3) (shear strength greater than approximately 90 kPa). It is appropriate for unsaturated subgrade soils. The primary function of a geotextile in this application is separation.

7.3.3 Geotextile Requirements. The geotextile shall meet the requirements of Table 3. All numeric values in Table 3 except AOS represent MARV in the weakest principal direction. Values for AOS represent maximum average roll values.

7.3.4 The property values in Table 3 represent default values which provide for sufficient geotextile survivability under most construction conditions. Note 1 of Table 3 provides for a reduction in the minimum property requirements when sufficient survivability information is available. The Engineer may also specify properties different from those listed in Table 3 based on engineering design and experience.

7.4 Stabilization Requirements

7.4.1 Description. This specification is applicable to the use of a geotextile in wet, saturated conditions to provide the coincident functions of separation and filtration. In some installations, the geotextile can also provide the function of reinforcement. Stabilization is applicable to pavement structures constructed over soils with a California Bearing Ratio between one and three (1 < CBR < 3) (shear strength between approximately 30 kPa and 90 kPa).

7.4.2 The stabilization application is appropriate for subgrade soils which are saturated due to a high groundwater table or due to prolonged periods of wet weather. This specification is not appropriate for embankment reinforcement where stress conditions may cause global subgrade foundation or stability failure. Reinforcement of the pavement section is a site specific design issue.

7.4.3 Geotextile Requirements. The geotextile shall meet the requirements of Table 4. All numeric values in Table 4 except AOS represent MARV in the weakest principal direction. Values for AOS represent maximum average roll values.

7.4.4 The property values in Table 4 represent default values which provide for sufficient geotextile survivability under most construction conditions. Note 1 of Table 4 provides for a reduction in the minimum property requirements when sufficient survivability information is available. The Engineer may also specify properties different from those listed in Table 3 based on engineering design and experience.

7.5 Permanent Erosion Control

7.5.1 Description. This specification is applicable to the use of a geotextile between energy absorbing armor systems and in the in-situ soil to prevent soil loss resulting in excessive scour and to prevent hydraulic uplift pressures causing instability of the permanent erosion control system. This specification does not apply to other types of geosynthetic soil erosion control materials such as turf reinforcement mats.

7.5.2 The primary function the geotextile serves in permanent erosion control applications is filtration. Geotextile filtration properties are a function of hydraulic conditions, and in-situ soil gradation, density and plasticity.

7.5.3 Geotextile Requirements. The geotextile shall meet the requirements of Table 5. Woven slit film geotextiles (i.e., geotextiles made from yarns of flat, tape-like character) will not be allowed. All numeric values in Table 5 except AOS represent MARV in the weaker principal direction. Values for AOS represent maximum average roll values.

7.5.4 The property values in Table 5 represent default values which provide for sufficient geotextile survivability under conditions similar to or less severe than those described under Note 2 of Table 5. Note 3 of Table 5 provides for a reduction in the minimum property requirements when sufficient survivability information is available or when the potential for construction damage is reduced. The Engineer may also specify properties different from those listed in Table 5 based on engineering design and experience.

8. TEMPORARY SILT FENCE REQUIREMENTS

8.1 Description. This specification is applicable to the use of a geotextile as a vertical, permeable interceptor designed to remove suspended soil from overland water flow. The function of a temporary silt fence is to filter and allow settlement of soil particles form sediment laden water. The purpose is t prevent the eroded soil from being transported off the construction site by water runoff.

8.2 Geotextile Requirements. The geotextile used for temporary silt fence may or may not be supported between posts with wire or polymeric mesh. The temporary silt fence geotextile shall meet the requirements of Table 6. All numeric values in Table 6 except AOS represent MARV. Values for AOS represent maximum average roll values.

8.3 Field monitoring shall be performed to verify that the armor system placement does not damage the geotextile. The minimum height above ground for all silt fence shall be 760 mm. Minimum embedment depth shall be 150 mm. Refer to Appendix for more detailed installation requirements.

9. PAVING FABRIC REQUIREMENTS

9.1 Description. This specification is applicable to the use of a paving fabric, saturated with asphalt cement, between pavement layers. The function of the paving fabric is to act as a waterproofing and stress relieving membrane within the pavement structure. This specification is not intended to describe fabric membrane systems specifically designed for pavement joints and localized (spot) repairs.

9.2 Paving Fabric Requirements. The paving fabric shall meet the requirements of Table 7. All numeric values in Table 7 represent MARV in the weaker principal direction.

Table 1. Geotextile Strength Property Requirements

			Geotextile Class[1]					
			Class 1		Class 2		Class 3	
	Test Methods	Units	Elongation $< 50\%$[2]	Elongation $\geq 50\%$[2]	Elongation $< 50\%$[2]	Elongation $\geq 50\%$[2]	Elongation $< 50\%$[2]	Elongation $\geq 50\%$[2]
Grab Strength	ASTM D 4632	N	1400	900	1100	700	800	500
Sewn Seam Strength[3]	ASTM D 4632	N	1260	810	990	630	720	450
Tear Strength	ASTM D 4533	N	500	350	400[4]	250	300	180
Puncture Strength	ASTM D 4833	N	500	350	400	250	300	180
Burst Strength	ASTM D 3786	kPa	3500	1700	2700	1300	2100	950
Permittivity	ASTM D 4991	sec^{-1}	Minimum property values for permittivity, AOS, and UV stability are based on geotextile application. Refer to Table 2 subsurface drainage, Table 3 for separation, Table 4 for stabilization, and Table 5 for permanent erosion control.					
Apparent Opening Size	ASTM D 4751	mm						
Ultraviolet Stability	ASTM D 4355	%						

Property Notes for Table 1

1 Required geotextile class is designated in Table 2, 3, 4, or 5 for the indicated application. The severity of installation conditions for the application generally dictate the required geotextile class. Class 1 is specified for more severe or harsh installation conditions where there is a greater potential for geotextile damage, and Class 2 and 3 are specified for less severe conditions.

2 As measured in accordance wit ASTM D 4632.

3 When sewn seams are required. Refer to Appendix for overlap seam requirements.

4 The required MARV tear strength for woven monofilament geotextiles is 250 N.

Table 2. Subsurface Drainage Geotextile Requirements

	Test Methods	Units	Requirements Percent In-Situ Soil Passing 0.075 mm[1] < 15	15 to 50	> 50
Geotextile Class			Class 2 from Table 1[2]		
Permittivity[3,4]	ASTM D 4491	sec^{-1}	0.5	0.2	0.1
Apparent Opening Size[3,4]	ASTM D 4751	mm	0.43 max. avg. roll value	0.25 max. avg. roll value	0.22[5] max. avg. roll value
Ultraviolet Stability (Retained Strength)	ASTM D 4355	%	50% after 500 hrs of exposure		

Property Notes for Table 2

1 Based on grain size analysis of in-situ soil in accordance with AASHTO T 88.

2 Default geotextile selection. The Engineer may specify a Class 3 geotextile from Table 1 for trench drain applications based on one or more of the following:

a) The Engineer has found Class 3 geotextiles to have sufficient survivability based on field experience.

b) The Engineer has found class 3 geotextiles to have sufficient survivability based on laboratory testing and visual inspection of a geotextile sample removed from a field test section constructed under anticipated field conditions.

c)Subsurface drain depth is less than 2 m, drain aggregate diameter is less than 30 mm and compaction requirement is less than 95% of AASHTO T 99.

3 These default filtration property values are based on the predominant particle size of in-situ soil. In addition to the default permittivity value, the Engineer may require geotextile permeability and/or performance testing based on engineering design for drainage systems in problematic soil environments.

4 Site specific geotextile design should be performed especially if one or more of the following problematic soil environments are encountered: unstable or highly erodible soils such as non-cohesive silts; gap graded soils; alternating sand/silt laminated soils; dispersive clays; and/or rock flour.

5 For cohesive soils with a plasticity index greater than 7, geotextile maximum average roll values for apparent opening size is 0.30 mm.

Table 3. Separation Geotextile Property Requirements

	Test Methods	Units	Requirements
Geotextile Class			Class 2 from Table 1[1]
Permittivity	ASTM D 4491	sec^{-1}	0.02[2]
Apparent Opening Size	ASTM D 4751	mm	0.60 max. avg. roll value
Ultraviolet Stability (Retained Strength)	ASTM D 4355	%	50% after 500 hrs of exposure

Property Notes for Table 3

1 Default geotextile selection. The Engineer may specify a Class 3 geotextile from Table 1 based on one or more of the following:
   a) The Engineer has found Class 3 geotextiles to have sufficient survivability based on field experience.
   b) The Engineer has found class 3 geotextiles to have sufficient survivability based on laboratory testing and visual inspection of a geotextile sample removed from a field test section constructed under anticipated field conditions.
   c) Aggregate cover thickness of the first lift over the geotextile exceeds 300 m and the aggregate diameter is less than 50 mm.

2 Default value. Permittivity of the geotextile should be greater than that of the soil ($\psi_g > \psi_s$). The Engineer may also require the permeability of the geotextile to be greater than that of the soil ($k_g > k_s$).

Table 4. Stabilization Geotextile Property Requirements

	Test Methods	Units	Requirements
Geotextile Class			Class 1 from Table 1[1]
Permittivity	ASTM D 4491	sec^{-1}	0.05[2]
Apparent Opening Size	ASTM D 4751	mm	0.43 max. avg. roll value
Ultraviolet Stability (Retained Strength)	ASTM D 4355	%	50% after 500 hrs of exposure

Property Notes for Table 4

1 Default geotextile selection. The Engineer may specify a Class 2 or 3 geotextile from Table 1 based on one or more of the following:
   a) The Engineer has found the class of geotextile to have sufficient survivability based on field experience.
   b) The Engineer has found the class of geotextile to have sufficient survivability based on laboratory testing and visual inspection of a geotextile sample removed from a field test section constructed under anticipated field conditions.

2 Default value. Permittivity of the geotextile should be greater than that of the soil ($\psi_g > \psi_s$). The Engineer may also require the permeability of the geotextile to be greater than that of the soil ($k_g > k_s$).

Table 5. Permanent Erosion Control Geotextile Requirements

	Test Methods	Units	Requirements Percent In-Situ Soil Passing 0.075 mm[1] < 15	15 to 50	> 50
Geotextile Class					
Woven Monofilament Geotextiles				Class 2 from Table 1[2]	
All Other Geotextiles				Class 1 from Table 1[2,3]	
Permittivity[1,4]	ASTM D 4491	sec^{-1}	0.7	0.2	0.1
Apparent Opening Size[3,4]	ASTM D 4751	mm	0.43 max. avg. roll value	0.25 max. avg. roll value	0.22[5] max. avg. roll value
Ultraviolet Stability (Retained Strength)	ASTM D 4355	%		50% after 500 hrs of exposure	

Property Notes for Table 5

1 Based on grain size analysis of in-situ soil in accordance with AASHTO T 88.

2 As a general guideline, the default geotextile selection is appropriate for conditions of equal or less severity than either of the following:

a) Armor layer stone weights do not exceed 100 kg, stone drop height is less than 1 m, and no aggregate bedding is required.

b) Armor layer stone weights do not exceed 100 kg, stone drop height is less than 1 m, and the geotextile is protected by a 150 mm thick aggregate bedding layer designed to be compatible with the armor layer. More severe applications require an assessment of geotextile survivability based on a field trial section and may require a geotextile with higher strength properties.

3 The Engineer may specify a Class 2 geotextile from Table 1 based on one or more of the following:

a) The Engineer has found Class 2 geotextiles to have sufficient survivability based on field experience.

b) The Engineer has found Class 2 geotextiles to have sufficient survivability based on laboratory testing and visual inspection of a geotextile sample removed from a field test section constructed under anticipated field conditions.

c) Armor layer stone weights do not exceed 100 kg, stone drop height is less than 1 m, and the geotextile is protected by a 150 mm thick aggregate bedding layer designed to be compatible with the armor layer.

d) Armor layer stone weights do not exceed 100 kg, and stone is placed with a zero drop height.

4 These default filtration property values are based on the predominant particle size of in-situ soil. In addition to the default permittivity value, the Engineer may require geotextile permeability and/or performance testing based on engineering design for drainage systems in problematic soil environments.

5a Site specific geotextile design should be performed especially if one or more of the following problematic soil environments are encountered: unstable or highly erodible soils such as non-cohesive silts; gap graded soils; alternating sand/silt laminated soils; dispersive clays; and/or rock flour.

5b For cohesive soils with a plasticity index greater than 7, geotextile maximum average roll values for apparent opening size is 0.30 mm.

Table 6. Temporary Silt Fence Property Requirements

	Test Methods	Units	Requirements: Supported Silt Fence[1]	Requirements: Unsupported Silt Fence, Geotextile Elongation ≥ 50%[2]	Requirements: Unsupported Silt Fence, Geotextile Elongation < 50%[2]
Maximum Post Spacing			1.2 m	1.2 m	2 m
Grab Strength	ASTM D 4632	N			
Machine Direction			400	550	550
X-Machine Direction			400	450	450
Permittivity[3]	ASTM D 4491	sec^{-1}	0.05	0.05	0.05
Apparent Opening Size	ASTM D 4751	mm	0.60 max. avg. roll value	0.60 max. avg. roll value	0.60 max. avg. roll value
Ultraviolet Stability (Retained Strength)	ASTM D 4355	%	70% after 500 hrs of exposure		

Property Notes for Table 6

1 Silt fence support shall consist of 14 gage steel wire with a mesh spacing of 150 mm by 150 mm or prefabricated polymeric mesh of equivalent strength.

2 As measured in accordance with ASTM D 4632.

3 These default filtration property values are based on empirical evidence with a variety of sediments. For environmentally sensitive areas, a review of previous experience and/or site or regionally specific geotextile tests should be performed by the agency to confirm suitability of these requirements.

Table 7. Paving Fabric Property Requirements

	Test Methods	Units	Requirements
Grab Strength	ASTM D 4632	N	450
Mass Per Unit Area	ASTM D 3776	gm/m^2	140
Ultimate Elongation	ASTM D 4632	%	≥ 50
Asphalt Retention[1]	Texas DOT Item 3009	l/m^2	Notes 1 and 2
Melting Point	ASTM D 276	°C	150

Property Notes for Table 7

1 Asphalt required to saturate paving fabric only. Asphalt retention must be provided in manufacturer certification (refer to Section 4). Value does not indicate the asphalt application rate required for construction. Refer to Appendix for discussion of asphalt application rate.

2 Product asphalt retention property must meet the MARV value provided by the manufacturer certification (refer to Section 4).

APPENDIX

CONSTRUCTION/ INSTALLATION GUIDELINES

A.1 GENERAL

A1.1 This Appendix is intended fro use in conjunction with AASHTO Specification M 288-96 for Geotextiles. The Specification details materials properties for geotextiles used in drainage, erosion control, separation/stabilization, silt fences, and pavement overlay application. The material properties are only one factor in a successful installation involving geotextiles. Proper construction and installation techniques are essential in order to ensure hat the intended function of the geotextile is fulfilled.

A1.2 Geotextile Identification, Packaging, and Storage

A1.2.1 Refer to ASTM D 4873.

A1.3 Geotextile Exposure Following Placement

A1.3.1 Atmospheric exposure of geotextiles to the elements following lay down shall be a maximum of 14 days to minimize damage potential.

A4.1 Seaming

A1.4.1 If a sewn seam is to be used for the seaming of the geotextile, the thread used shall consist of high strength polypropylene, or polyester. Nylon thread shall not be used. For erosion control applications, the thread shall also be resistant to ultraviolet radiation. The thread shall be of contrasting color to that of the geotextile itself.

A1.4.2 For seams which are sewn in the field, the Contractor shall provide at least a 2 meter length of sewn seam for sampling by the Engineer before the geotextile is installed. For seams which are sewn in the factory, the Engineer shall obtain samples of the factory seams at random form any roll of geotextile which is used on the project.

A1.4.2.1 For seams that are field sewn, the seams sewn for sampling shall be sewn using the same equipment and procedures as will be used for the production seams. If seams are sewn in both the machine ane cross machine direction, samples of seams from both directions shall be provided.

A1.4.2.2 The seam assembly description shall be submitted by the Contractor along with the sample of the seam. The description shall include the seam type, stitch type, sewing thread, and stitch density.

A.2 DRAINAGE GEOTEXTILES[3] (See Specification Sections 7.1 & 7.2)

A2.1 Construction

A2.1.1 Trench excavation shall be done in accordance with details of the project plans. In all instances excavation shall be done in such a way so as to prevent large voids from occurring in the sides and bottom of the trench. The graded surface shall be smooth and free of debris.

A2.1.2 In the placement of the geotextile for drainage applications, the geotextile shall be placed loosely with no wrinkles or folds, and with no void spaces between the geotextile and

[3] Geotextiles used as sheet drains are not included in the discussions in this section.

the ground surface. Successive sheets of geotextiles shall be overlapped a minimum of 300 mm, with the upstream sheet overlapping the downstream sheet.

A2.1.2.1 In trenches equal to or greater than 300 mm in width, after placing the drainage aggregate the geotextile shall be folded over the top of the backfill material in a manner to produce a minimum overlap of 300 mm. In trenches less than 300 mm but greater than 100 mm wide, the overlap shall be equal to the width of the trench. Where the trench is less than 100 mm the geotextile overlap shall be sewn or otherwise bonded. All seams shall be subject to the approval of the Engineer.

A2.1.2.2 Should the geotextile be damaged during installation or drainage aggregate placement, a geotextile patch shall be placed over the damaged area extending beyond the damaged area a distance of 300 mm, or the specified seam overlap, whichever is greater.

A2.1.3 Placement of drainage aggregate should proceed immediately following placement of the geotextile. The geotextile should be covered with a minimum of 300 mm of loosely placed aggregate prior to compaction. If a perforated collection pipe is to be installed in the trench, a bedding layer of drainage aggregate should be placed below the pipe, with the remainder of the aggregate placed to the minimum required construction depth.

A2.1.3.1 The aggregate should be compacted with vibratory equipment to a minimum of 95 percent Standard AASHTO density unless the trench is required for structural support. If higher compactive effort is required, a Class 1 geotextile as per Table 1 of M 288 Specification is needed.

A2.1.4 Figures A1 through A3 illustrate various geotextile drainage application details.

A.3 SEPARATION/ STABILIZATION GEOTEXTILES (See Specification Sections 7.1, 7.3 and 7.4)

A3.1 Construction

A3.1.1 The installation site shall be prepared by clearing, grubbing, and excavation or filling the area to the design grade. This includes removal of top soil and vegetation.

NOTE 1— Soft spots and unsuitable areas will be identified during site preparation or subsequent proof rolling. These areas shall be excavated and backfilled with select material and compacted using normal procedures.

A3.1.2 The geotextile shall be laid smooth without wrinkles or folds on the prepared subgrade in the direction of construction traffic. Adjacent geotextile rolls shall be overlapped, sewn or joined as required in the plans. Overlaps shall be in the direction as shown on the plans. See Table A1 for overlap requirements.

TABLE A1

SOIL CBR	MINIMUM OVERLAP
Greater than 3	300 - 450 mm
1 - 3	0.6 - 1 m
0.5 - 1	1 m or sewn
Less than 0.5	Sewn
All Roll Ends	1 m or sewn

A3.1.2.1 On curves the geotextile may be folded or cut to conform to the curves. The fold or overlap shall be in the direction of construction and held in place by pins, staples, or piles of fill or rock.

A3.1.2.2 Prior to covering, the geotextile shall be inspected by a certified inspector of the Engineer to ensure that the geotextile has not been damaged (i.e., holes, tears, rips) during installation. Damaged geotextiles, as identified by the Engineer, shall be repaired immediately. Cover the damaged area with a geotextile patch which extends an amount equal to the required overlap beyond the damaged area.

A3.1.3 The subbase shall be placed by end dumping onto the geotextile from the edge of the geotextile, or over previously placed subbase aggregate. Construction vehicles shall not be allowed directly on the geotextile. The subbase shall be placed such that at least the minimum specified lift thickness shall be between the geotextile and equipment tires or tracks at all times. Turning of vehicles shall not be permitted on the first lift above the geotextile.

NOTE 2 — On subgrades having a CBR value of less than 1, the subbase aggregate should be spread in its full thickness as soon as possible after dumping to minimize the potential of localized subgrade failure due to overloading of the subgrade.

A3.1.3.1 Any ruts occurring during construction shall be filled with additional subbase material, and compacted to the specified density.

A3.1.3.2 If placement of the backfill material causes damage to the geotextile, the damaged area shall be repaired as previously described in section A2.1.3.1. The placement procedures shall be then be modified to eliminate further damage from taking place (i.e., increase initial lift thickness, decrease equipment loads, etc.).

NOTE 3 — In stabilization applications, the use of vibratory compaction equipment is not recommended with the initial lift of subbase material, as it may cause damage to the geotextile.

A4. EROSION CONTROL GEOTEXTILES (See Specification Section 7.5.)

A4.1 Construction

A4.1.1 The geotextile shall be placed in intimate contact with the soils without wrinkles or folds and anchored on a smooth graded surface approved by the Engineer. The geotextile shall be placed in such a manner that placement of the overlying materials will not excessively stretch so as to tear the geotextile. Anchoring of the terminal ends of the geotextile shall be accomplished through the use of key trenches or aprons at the crest and toe of slope. Refer to Figures A4 through A7 for construction details.

NOTE 1 — In certain applications to expedite construction, 450 mm anchoring pins placed on 600 to 1800 mm centers, depending on the slope of the covered area, have been used successfully.

A4.1.1.1 The geotextile shall be placed with the machine direction parallel to the direction of water flow which is normally parallel to the slope for erosion control runoff and wave action (see Figure A4), and parallel to the stream or channel in the case of streambank and channel protection (see Figure A6). Adjacent geotextile sheets shall be joined by either sewing or overlapping. Overlapped seams of roll ends shall be a minimum of 300 mm except where placed under water.

In such instances the overlap shall be a minimum of 1 m. Overlaps of adjacent rolls shall be a minimum of 300 mm in all instances.

NOTE 2 — When overlapping, successive sheets of the geotextile shall be overlapped upstream over downstream, and/or upslope over downslope. In cases where wave action or multidirectional flow is anticipated, all seams perpendicular to the direction of flow shall be sewn.

A4.1.1.2 Care shall be taken during installation so as to avoid damage occurring to the geotextile as a result of the installation process. Should the geotextile be damaged during installation, a geotextile patch shall be placed over the damaged area extending 1 m beyond the perimeter of the damage.

A4.1.2 The armor system placement shall begin at the toe and proceed up the slope. Placement shall take place so as to avoid stretching and subsequent tearing of the geotextile. Riprap and heavy stone filling shall not be dropped from a height of more than 300 mm. Stone with a mass of more than 100 kg shall not be allowed to roll down the slope.

A4.1.2.1 Slope protection and smaller sized of stone filling shall not be dropped from a height exceeding 1 m, or a demonstration provided showing that the placement procedures will not damage the geotextile. In underwater applications, the geotextile and backfill material shall be placed the same day. All void spaces in the armor stone shall be backfilled with small stone to ensure full coverage.

A4.1.2.2 Following placement of the armor stone, grading of the slope shall not be permitted if the grading results in movement of the stone directly above the geotextile.

A4.1.3 Field monitoring shall be performed to verify that the armor system placement does not damage the geotextile.

4.1.3.1 Any geotextile damaged during backfill placement shall be replaced as directed by the Engineer, at the contractor's expense.

A5. SILT FENCE GEOTEXTILES (See Specification Section 8.)

A5.1 Related Material Requirements

A5.1.1 Wood, steel, or synthetic support posts having a minimum length of 1 m plus the burial depth may be used. They shall be of sufficient strength to resist damage during installation and to the support applied loads due to material build up behind the silt fence.

NOTE 1 —It has been found that hardwood post having dimensions of at least 30 mm x 30 mm, No. 2 Southern Pine at least 65 mm x 65 mm or steel posts of U, T, L, or C shape, weighing 600 g per 300 mm have performed satisfactorily.

A5.1.2 Wire or polymer support fence shall be at least 750 mm high and strong enough to support applied loads. Polymer support fences shall meet the same ultraviolet degradation requirements as the geotextile.

NOTE 2 —Wire support fences having at least 6 horizontal wires, and being at least 14 gauge wire have performed satisfactorily. Vertical wires should be a maximum of 150 mm apart.

A5.2 Construction

A5.2.1 The geotextile at the bottom of the fence shall be buried in a "J" configuration to a minimum depth of 150 mm in a trench so that no flow can pass under the silt fence. The trench shall be backfilled and the soil compacted over the geotextile.

A5.2.1.1 The geotextile shall be spliced together with a sewn seam only at a support post, or two sections of fence may be overlapped instead.

A5.2.1.2 The Contractor must demonstrate to the satisfaction of the Engineer that the geotextile can withstand the anticipated sediment loading.

A5.2.1.3 See Figure A8 for details.

A5.2.2 The posts shall be placed at a spacing as shown on the project plans. Posts should be driven or placed a minimum of 500 mm into the ground. Depth shall be increased to 600 mm if fence is placed on a slope of 3:1 or greater.

NOTE 3 —Where 500 mm depth is impossible to attain, the posts should be adequately secured to prevent overturning of the fence due to sediment loading.

A5.2.3 The support fence shall be fastened securely to the upslope side of the fence post. The support fence shall extend from the ground surface to the top of the geotextile.

A5.2.4 When self-supported fence is used, the geotextile shall be securely fastened to fence posts.

A5.2.5 Silt fences should be continuous and transverse to the flow. The silt fence should follow the contours of the site as closely as possible. The fence shall also be placed such that the water cannot runoff around the end of the fence.

A5.2.5.1 The silt fence should be limited to handle an area equivalent to 90 square meters per 3 meters of fence. Caution should be used where the site slope is greater than 1:1, and water flow rates exceed 3 liters per second per 3 meters of fence.

A5.3 Maintenance

A5.3.1 The Contractor shall inspect all temporary silt fences immediately after each rainfall, and at least daily during prolonged rainfall. Any deficiencies shall be immediately corrected by the Contractor.

A5.3.1.1 The Contractor shall also make a daily review of the location of silt fences in areas where construction activities have altered the natural contour and drainage runoff to ensure that the silt fences are properly located for effectiveness. Where deficiencies exist as determined by the Engineer, additional silt fence shall be installed as directed by the Engineer.

A5.3.1.2 Damaged or otherwise ineffective silt fences shall be repaired or replaced promptly.

A5.3.2 Sediment deposits shall either be removed when the deposit reaches half the height of the fence, or a second silt fence shall be installed as directed by the Engineer.

A5.3.3 The silt fence shall remain in place until the Engineer directs it be removed. Upon removal, the Contractor shall remove and dispose of any excess sediment accumulations, dress the area to give it a pleasing appearance, and vegetate all bare areas in accordance with contract requirements.

A5.3.3.1 Removed silt fence may be used at other locations provided the geotextile and other material requirements continue to be met to the satisfaction of the Engineer.

A6. PAVING GEOTEXTILES
(See Specification Section 9.)

A6.1 Materials

A6.1.1 The sealant material used to impregnate and seal the geotextile, as well as bond it to both the base pavement and overlay, shall be a paving grade asphalt recommended by the geotextile manufacturer, and approved by the Engineer.

A6.1.1.1 Uncut asphalt cements are the preferred sealants; however, cationic and anionic emulsions may be used provided the precautions outlined in Section A6.3.3 are followed. Cutbacks and emulsions which contain solvents shall not be used.

A6.1.1.2 The grade of asphalt cement specified for hot-mix design in each geographic location is generally the most acceptable material.

A6.1.2 Washed concrete sand may be spread over an asphalt saturated geotextile to facilitate movement of equipment during construction or to prevent tearing or delamination of the geotextile. Hot-mix broadcast in front of construction vehicle tires may also be used to serve this purpose. If sand applied, excess quantities shall be removed from the geotextile prior to placing the surface course.

A6.1.2.1 Sand is not usually required. However, ambient temperatures are occasionally sufficiently high to cause bleed-through of the asphalt sealant resulting in undesirable geotextile adhesion to construction vehicle tires.

A6.2 Equipment

A6.2.1 The asphalt distributor shall be capable of spraying the asphalt sealant at the prescribed uniform application rate. No streaking, skipping, or dripping will be permitted. The distributor shall also be equipped with a hand spray having a single nozzle and positive shut-off valve.

A6.2.2 Mechanical or manual lay down equipment shall be capable of laying the geotextile smoothly.

A6.2.3 The following miscellaneous equipment shall be provided: stiff bristle brooms or squeegees to smooth the geotextile; scissors or blades to cut the geotextile; brushes for applying asphalt sealant to geotextile overlaps.

A6.2.4 Pneumatic rolling equipment to smooth the geotextile into the sealant, and sanding equipment may be required for certain jobs. Rolling is especially required on jobs where thin lifts or chip seals are being placed. Rolling helps ensure geotextile bond to the adjoining pavement layers in the absence of heat and weight associated with thicker lifts of asphaltic pavement.

A6.3 Construction

A6.3.1 Neither the asphalt sealant nor the geotextile shall be placed when weather conditions, in the opinion of the Engineer, are not suitable. Air and pavement temperatures shall be sufficient to allow the asphalt sealant to hold the geotextile in place. For asphalt cements, air temperature shall be 10°C and rising. For asphalt emulsions, air temperature shall be 15°C and rising.

A6.3.2 The surface on which the geotextile is to be placed shall be reasonably fress of dirt, water, vegetation, or other debris. Cracks exceeding 3 mm in width shall be filled with a suitable crack filler. Potholes shall be properly repaired as directed by the Engineer. Fillers shall be allowed to cure prior to geotextile placement.

A6.3.3 The specified rate of asphalt sealant application must be sufficient to satisfy the asphalt retention properties of the geotextile, and bond the geotextile and overlay to the old pavement.

NOTE 1 —When emulsions are used, the application rate must be increased to offset water content of the emulsion.

A6.3.3.1 Application of the sealant shall be by distributor spray bar, with hand spraying kept to a minimum. Temperature of the asphalt sealant shall be sufficiently high to permit uniform spray pattern. For asphalt cements the minimum temperature shall be 145°C. To avoid damage to the geotextile, however, the distributor tank temperature shall not exceed 160°C.

A6.3.3.2 Spray patterns for asphalt emulsion are improved by heating. Temperatures in the 55°C to 70°C range are desirable. A temperature of 70°C shall not be exceeded since higher temperatures may break the emulsion.

A6.3.3.3 The target width of asphalt sealant applications shall be the geotextile width plus 150 mm. The asphalt sealant shall not be applied any farther in advance of geotextile placement than the distance the contractor can maintain free of traffic.

A6.3.3.4 Asphalt spills shall be cleaned from the road surface to avoid flushing and geotextile movement.

A6.3.3.5 When asphalt emulsions are used, the emulsion shall be cured prior to placing the geotextile and final wearing surface. This means essentially no moisture remaining.

A6.3.4 The geotextile shall be placed onto the asphalt sealant with minimum wrinkling prior to the time the asphalt has cooled and lost tackiness. As directed by the engineer, wrinkles or folds in excess of 25 mm shall be slit and laid flat.

A6.3.4.1 Brooming and/or pneumatic rolling will be required to maximize geotextile contact with the pavement surface.

A6.3.4.2 Overlap of geotextile joints shall be sufficient to ensure full closure of the joint, but should not exceed 150 mm. Transverse joints shall be lapped in the direction of paving to prevent edge pickup by the paver. A second application of asphalt sealant to the geotextile overlaps will be required if in the judgement of the Engineer additional asphalt sealant is needed to ensure proper bonding of the double geotextile layer.

A6.3.4.3 Removal and replacement of geotextile that is damaged will be the responsibility of the contractor.

NOTE 2 — The problems associated with wrinkles are related to thickness of the asphalt lift being placed over the geotextile. When wrinkles are large enough to be folded over, there usually is not enough asphalt available from the tack coat to satisfy the requirement of multiple layers of geotextiles. Therefore, wrinkles should be slit and laid flat. Sufficient asphalt sealant should be sprayed on the top of the geotextile to satisfy the requirement of the lapped geotextile.

NOTE 3 —In overlapping adjacent rolls of geotextile it is desirable to keep the lapped dimension as small as possible and still provide a positive overlap. If the lapped dimension becomes too large, the problem of inadequate tack to satisfy the two lifts of geotextile and the old pavement may occur. If this problem does occur then additional asphaltic sealant should be added t the lapped areas. In the application of the additional sealant, care should be taken not to apply too much since an excess will cause flushing.

A6.3.4.4 Trafficking the geotextile will be permitted for emergency and construction vehicles only.

A6.3.5 Placement of the hot-mix overlay should closely follow geotextile laydown. The temperature of the mix shall not exceed 160°C. In the event asphalt bleeds through the geotextile causing construction problems before the overlay is placed, the affected areas shall be blotted by spreading sand. To avoid movement of, or damage to the seal-coat saturated geotextile, turning of the paver and other vehicles shall be gradual and kept to a minimum.

A6.3.6 Prior to placing a seal coat (or thin overlay such as an open-graded friction course), lightly sand the geotextile at a spread rate of 0.65 to 1 kg per m^2, and pneumatically roll the geotextile tightly into the sealant.

ADVISORY

It is recommended that for safety considerations, trafficking of the geotextile should not be allowed. However, if the contracting agency elects to allow trafficking, the following verbiage is recommended:
"If approved by the Engineer, the sealcoat saturated geotextile may be opened to traffic for 24 to 48 hours prior to installing the surface course. Warning signs shall be placed which advise the motorist that the surface may be slippery when wet. The signs shall also post the appropriate safe speed. Excess sand shall be broomed from the surface prior to placing the overlay. If, in the judgement of the Engineer, the fabric surface appears dry, and lacks tackiness following exposure to traffic, a light tack coat shall be applied prior to the overlay."

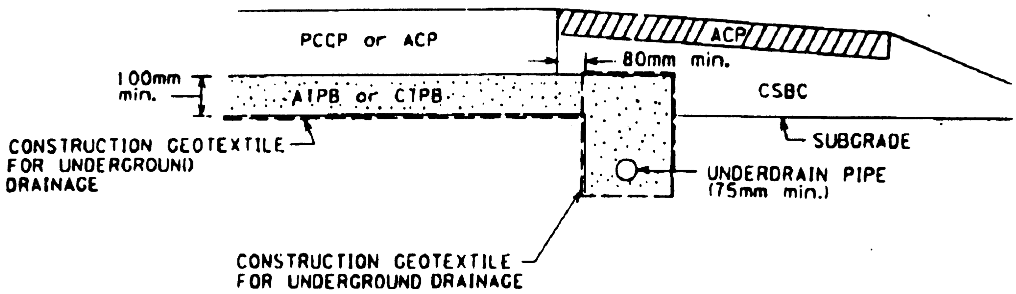

FIGURE A1 **Geotextile Drain Requirements for Permeable Bases**

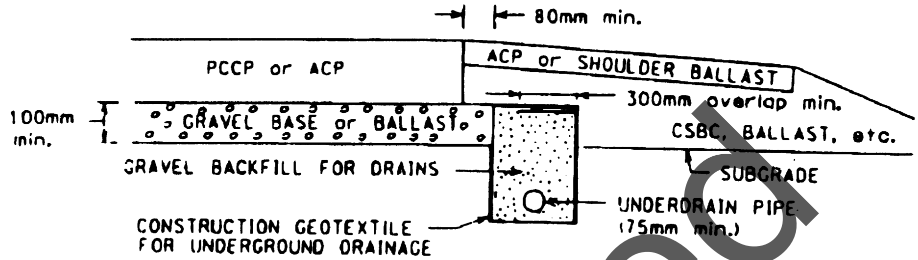

FIGURE A2 **Geotextile Wrapped Longitudinal Edge Drain**

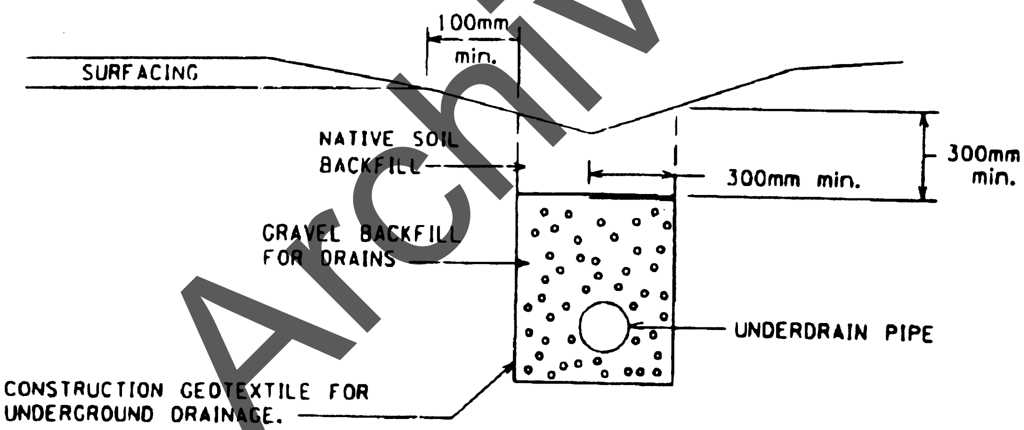

FIGURE A3 **Geotextile Wrapped Pavement Under Drain**

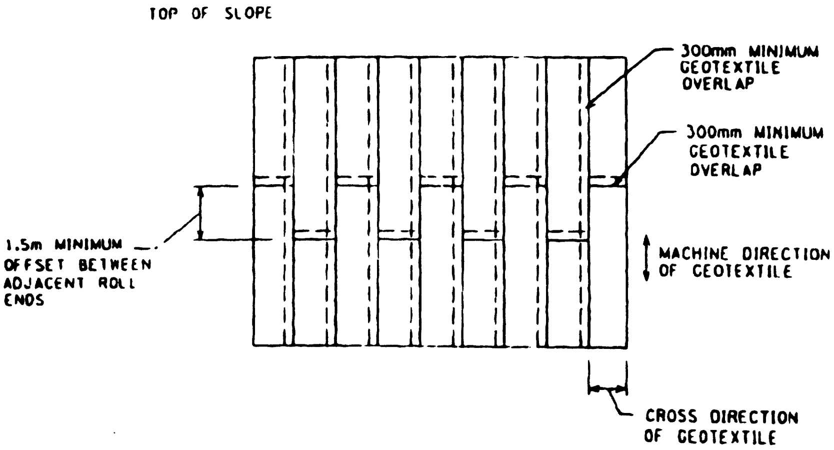

FIGURE A4 Method of Placing Geotextile for Protection of Cut and Fill Slopes

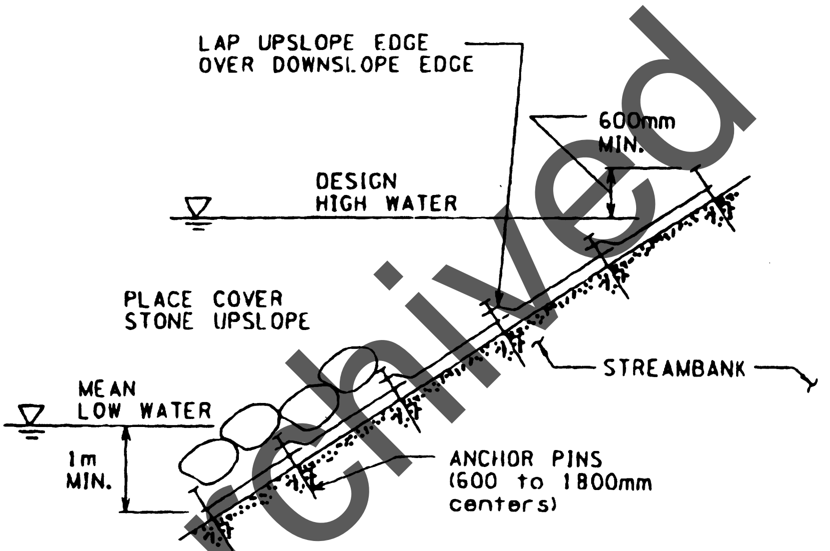

FIGURE A5 Cross Section of Slopes with Riprap

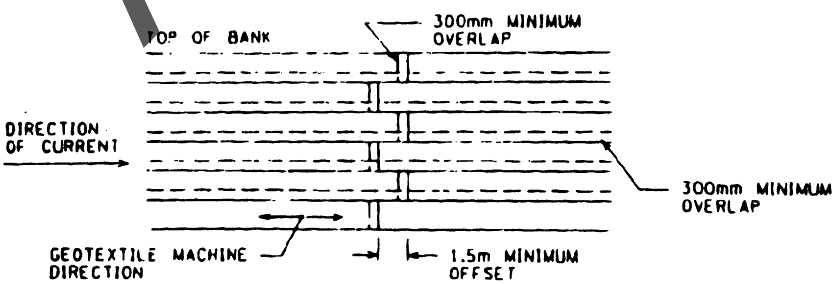

FIGURE A6 Geotextile Placement Scheme for Streambank Protection

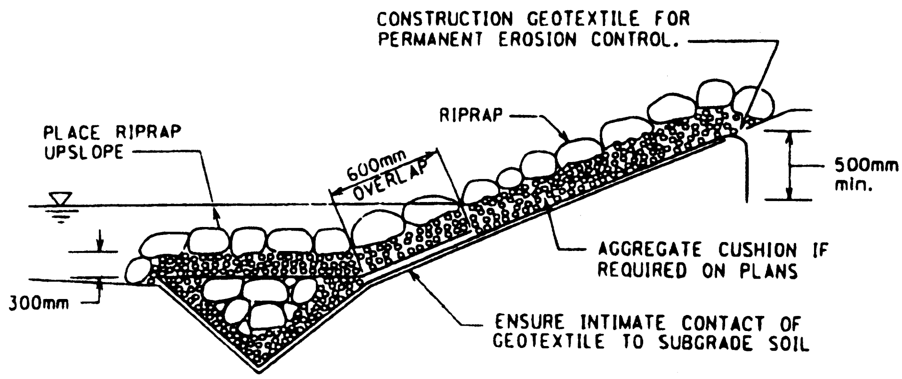

FIGURE A7 Key Detail at Top and Toe of Slope for Geotextiles Used for Permanent Erosion Control

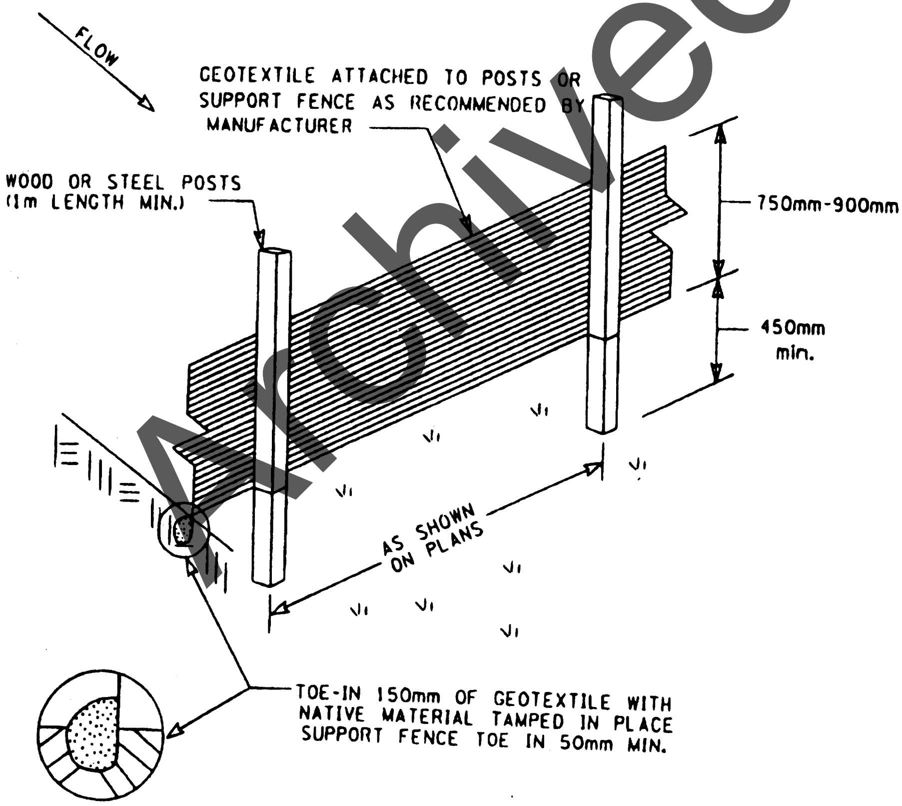

FIGURE A8 Typical Silt Fence Detail

Appendix E
GEOSYNTHETIC TEST STANDARDS

E-1 American Society for Testing and Materials

Endurance Properties

Specification for:

D 4886 Abrasion Resistance of Geotextiles (Sand/Sliding Block Method)
D 4355 Deterioration of Geotextiles from Exposure to Ultraviolet Light and Water (Xenon-Arc Type Apparatus)
D 4594 Effects of Temperature on Stability of Geotextiles
D 5322 Immersion Procedures for Evaluating the Chemical Resistance of Geosynthetics to Liquids

Test Methods for:

D 1987 Biological Clogging of Geotextile or Soil/Geotextile Filters
D 5397 Stress Crack Resistance of Polyolefin Geomembranes Using Notched Constant Tensile Load Test, Evaluation of
D 5262 Unconfined Tension Creep Behavior of Geosynthetics, Evaluating the
D 5596 Microscopic Evaluation of the Dispersion of Carbon Black in Polyolefin Geosynthetics
D 5885 Oxidative Induction Time of Polyolefin Geosynthetics by High Pressure Differential Scanning Calorimetry

Practice for:

D 5496 In Situ Immersion Testing of Geosynthetics
D 5721 Air-Oven Aging of Polyolefin Geomembranes
D 5747 Tests to Evaluate the Chemical Resistance of Geomembranes to Liquids

Guide for:

D 4873 Identification, Storage, and Handling of Geotextiles
D 5819 Selecting Test Methods for Experimental Evaluation of Geosynthetic Durability

Geomembranes

Specification for:

D 3083 Flexible Poly(Vinyl Chloride) Plastic Sheeting for Pond, Canal, and Reservoir Lining
D 4885 Performance Strength of Geomembranes by the Wide Strip Tensile Method, Determining
D 3020 Polyethylene and Ethylene Copolymer Plastic Sheeting for Pond, Canal, and Reservoir Lining

Practices for:

D 5323 2% Secant Modulus for Polyethylene Geomembranes, Determination of
D 4545 Integrity of Factory Seams Used in Joining Manufactured Flexible Sheet Geomembranes, Determining

D 4437 Integrity of Field Seams Used in Joining Flexible Polymeric Sheet Geomembranes, Determining
D 5641 Geomembrane Seam Evaluation by Vacuum Chamber
D 5820 Pressurized Air Channel Evaluation of Dual Seamed Geomembranes

Test Method for:
D 5494 Pyramid Puncture Resistance of Unprotected and Protected Geomembranes, Determination of
D 5514 Large Scale Hydrostatic Puncture Testing of Geosynthetics
D 5617 Multi-Axial Tension Test for Geosynthetics
D 5884 Determining the Tearing Strength of Internally Reinforced Geomembranes

Guide for:
D 5886 Selection of Test Methods to Determine the Rate of Fluid Permeation Through Geomembranes for Specific Applications

Mechanical Properties

Specification for:
D 4632 Breaking Load and Elongation of Geotextiles (Grab Method)
D 4833 Index Puncture Resistance of Geotextiles, Geomembranes, and Related Products
D 4884 Seam Strength of Sewn Geotextiles
D 4595 Tensile Properties of Geotextiles by the Wide Strip Method
D 4533 Trapezoid Tearing Strength of Geotextiles

Practices for:
D 4354 Sampling of Geosynthetics for Testing
D 4759 Specification Conformance of Geosynthetics, Determining
D 5818 Obtaining Samples of Geosynthetics from a Test Section for Assessment of Installation Damage

Test Method for:
D 5261 Measuring Mass Per Unit Area of Geotextiles
D 5321 Determining the Coefficient of Soil and Geosynthetic or Geosynthetic and Geosynthetic Friction by the Direct Shear Method

Permeability and Filtration

Test Method for:
D 4751 Apparent Opening Size of a Geotextile, Determining
D 5321 Coefficient of Soil and Geosynthetic or Geosynthetic and Geosynthetic Friction by the Direct Shear Method, Determining the
D 4716 Constant Head Hydraulic Transmissivity (In Plane Flow) of Geotextiles and Geotextile Related Products
D 5141 Filtering Efficiency and Flow Rate of a Geotextile for Silt Fence Application Using Site Specifications, Determine
D 5199 Measuring Nominal Thickness of Geotextiles and Geomembranes

D 5101 Measuring the Soil-Geotextile System Clogging Potential (By the Gradient Ratio)
D 5493 Permittivity of Geotextiles Under Load
D 4491 Water Permeability of Geotextiles by the Permittivity Method
D 5567 Hydraulic Conductivity Ratio (HCR) Testing of Soil/Geotextile Systems

Terminology

D 4439 Geosynthetics

Geosynthetic Clay Liners

Test Method for:

D 5887 Measurement of Index Flux Through Saturated Geosynthetic Clay Liner Specimens Using Flexible Wall Permeameter
D 5890 Swell Index of Clay Mineral Component of Geosynthetic Clay Liners
D 5891 Fluid Loss of Clay Component of Geosynthetic Clay Liners

Practices for:

D 5888 Storage and Handling of Geosynthetic Clay Liners
D 5889 Quality Control of Geosynthetic Clay Liners

New ASTM Standards Since Printing of This Manual

E-2 Geosynthetic Research Institute

GRI TEST METHODS & STANDARDS

Geotextile (GT) Related

GT1 Clogging Potential via Long-Term Flow Rate
GT2 Biological Clogging of Geotextile or Soil/Geotextile Filters (discontinued, see ASTM D 1987)
GT3 Deterioration of Geotextiles from Outdoor Exposure
GT4 Geotextile Permittivity-Under-Load (discontinued, see ASTM D 5493)
GT5 Tension Creep Testing of Geotextiles (discontinued, see ASTM D 5262)
GT6 Geotextile Pullout
GT7 Determination of Long-Term Design Strength of Geotextiles
GT8 Fine Fraction Filtration Using Geotextile Filters
GT 9 Grip Types for Use in Wide Width Testing of Geotextiles and Geogrids

Geogrid (GG) Related

GG1 Geogrid Rib Tensile Strength
GG2 Geogrid Junction Strength
GG3a Tension Creep Testing of Stiff Geogrids (discontinued, see ASTM D 5262)
GG3b Tension Creep Testing of Flexible Geogrids (discontinued, see ASTM D 5262)
GG4a Determination of the Long-Term Design Strength of Stiff Geogrids
GG4b Determination of the Long-Term Design Strength of Flexible Geogrids
GG5 Test Method for Geogrid Pullout
GG6 Grip Types for Use in Wide Width Testing of Geotextiles and Geogrids

Geonet (GN) Related

GN1 Compression Behavior of Geonets

Geomembrane (GM) Related

GM1 Seam Evaluation by Ultrasonic Shadow Method
GM2 Embedment Depth for Anchorage Mobilization
GM3 Large Scale Hydrostatic Puncture Test
GM4 Three Dimensional Geomembrane Tension Test (discontinued, see ASTM D 5617)
GM5a Notched Constant Tensile Load (NCTL) Test for Polyolefin Resins or Geomembranes (discontinued, see ASTM D 5397)
GM5b Single Point NCTL Test for Polyolefin Resin or Geomembranes (discontinue, see ASTM D 5397 Appendix)
GM5c Seam Constant Tensile Load (SCTL) Test for Polyolefin Geomembrane Seams
GM6 Pressurized Air Channel Test for Dual Seamed Geomembranes
GM7 Accelerated Curing of Geomembrane Test Strip Seams Made by Chemical Fusion Methods
GM 8 Measurement of the Core Thickness of Textured Geomembranes
GM9 Cold Weather Seaming of Geomembranes
GM10 Specification for Stress Crack Resistance of HDPE Geomembrane Sheet
GM11 Accelerated Weathering of Geomembranes Using a Fluorescent UVA Device
GM12 Asperity Measurement of Textured Geomembranes Using a Depth Gage

Geosynthetic Clay Liner (GCL) Related

GCL1 Swell Measurement of the Clay Component of GCL's
GCL2 Permeability of Geosynthetic Clay Liners (GCLs)

Geocomposite (GC) Related

GC1 Soil-Filter Core Combined Flow Test
GC2 Strip Drain Flow Rate Under Load
GC3 Strip Drain Kinking Efficiency
GC4 Compression Behavior of Prefabricated Edge Drains and Sheet Drains
GC5 Erosion Control Systems to Protect Against Soil Detachment by Rainfall Impact and Overload Flow Transport
GC6 Erosion Control Systems for High Velocity Flows in Channels

Geosynthetic (GS) Related (*i.e.*, Multipurpose)

GS1 CBR Puncture Strength
GS2 Rupture Strength by Pendulum Impact for Geotextiles-Geomembranes-Geocomposites
GS3 In-Situ Monitoring of the Mechanical Performance of Geosynthetics
GS4 Time Dependent (Creep) Deformation Under Normal Pressure
GS5 Impregnation of Geosynthetics Under Load
GS6 Interface Friction Determination by Direct Shear Testing (discontinued, see ASTM D 5321)
GS7 Determining the Index Friction Properties of Geosynthetics
GS8 Determining the Connection Strength of Mechanically Anchored Geosynthetics
GS9 Oxidative Induction Time of Polyethylene Geosynthetics by High Pressure Differential Scanning Calorimetry (discontinued, see ASTM D 5885)

New GRI Standards Since Printing of This Manual

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Appendix F
FHWA Geotechnology Technical Note
— Degradation Reduction Factors for Geosynthetics —

The information in this Note was taken from results of recent research performed under the FHWA structures research program.

The Note concerns one major aspect of the geosynthetic reinforcement issue dealing with the uncertainty of the geosynthetic material's long term durability features. Recent research results show that it is no longer a serious problem to predict the design service life for these materials, which has caused some reluctance by practitioners to specify them on highway construction projects.

Degradation reduction factors have been developed and are reported in the attached Technical Note, which can be used to determine appropriate allowable tensile strengths of commonly used geosynthetic materials that are specified for soil reinforcement projects. These scientifically determined factors are presented as a proposed improvement to current AASHTO default values which cause unnecessarily high material costs. The new values will save tens of millions of dollars per year.

This Technical Note was developed jointly by the members of the Contractor research team and the Geo-Technology Outreach Team. These teams consist of engineers and scientists from the Offices of Engineering R & D, Engineering and the key investigators for the contractor.

May 1, 1997

Federal Highway Administration Geotechnology Technical Note

From:
Geotechnology Outreach Team

Topic:
Degradation Reduction Factors for Geosynthetics

Description:
FHWA and AASHTO have spent considerable time and effort recently to develop degradation reduction factors for the determination of allowable tensile strength of geosynthetics used for soil reinforcement. Due to their economic advantages and relatively inert state, the use of polymeric materials as soil reinforcing members in Mechanically Stabilized Earth Walls (MSE), Reinforced Soil Slopes (RSS) and embankment foundation stabilization is increasing. In these applications the geosynthetic must maintain its tensile strength properties after exposure to construction stresses and exposure to an in-soil environment over the anticipated design life of 75 to 100 years. Because of their relatively short historical period in use, there is some reluctance by designers to specify them because there are some perceived uncertainties as to their durability.

Potential degradation of polymeric reinforcements (installation damage and aging) depends on the specific polymer, the configuration of the reinforcement, the environment to which they are exposed, and the stress level to which they are subjected. The current design approach to account for installation damage and long term degradation losses is to apply reduction factors to the ultimate tensile strength values. The allowable strength, T_{al} (in accordance with 1996 AASHTO specifications) is obtained from:

$$T_{al} = \frac{T_{ult}}{RF_{CR}\ RF_{ID}\ RF_{D}} = \frac{T_{ult}}{RF}$$

where the degradation induced reduction factor RF_{ID} is for installation damage and RF_D for aging. A total reduction factor (RF) is the product of all these individuals factors. The reduction factor for creep, RF_{CR} is an inherent time dependent property, not a degradation factor, as it does not alter any short term ultimate strength or the molecular weight of the polymer. Testing protocols to determine creep limiting strengths are specified in the 1997 AASHTO interims.

In the absence of specific data, it has been common practice to apply default reduction factors as high as 3 for installation damage, 2 for aging and from 2 to 5 for creep, or a total reduction factor (RF) of 7 under certain restrictive and defined conditions as specified in the 1997 AASHTO Interims. More current guidance has been developed from on-going FHWA sponsored research.[1,2]

Purpose:

This Technical Note is intended to summarize current research results and provide the basis for selecting aging (RF_D) and installation damage (RF_{ID}) reduction factors, consistent with the in-ground regime and construction methods specified, in lieu of using default reduction factors.

Cost Impacts:

The use of default degradation values reduces the allowable tensile strength and unnecessarily increases the cost of geosynthetic soil reinforcement materials. For instance, the application of current test information would permit on average a reduction of RF from 7 to 5. This decrease would increase the allowable strength by approximately 30%. The national impact, based on an estimated usage of 36 million m^2 on private and public projects for stabilization/reinforcement applications, is a potential cost savings of $30 million per year.

Variables in the Determination of Reduction Factors:

Installation Damage, RF_{ID}

Significant loss of tensile strength to geosynthetics can be partly attributed to mechanical or abrasion damage during the placement and compaction of backfill. This damage is not time dependent as it occurs concurrently with construction. The reduction in strength is measured by field installation damage testing in accordance with ASTM D 5818.

The variables which affect the level of damage for each geosynthetic include (1) gradation and angularity of backfill, (2) weight, manufacturing method and type of geosynthetic, (3) lift thickness and (4) the type and compactive effort used. The first two variables have been found to generally govern as the lift thickness for transportation related projects is always on the order of 250 to 300 mm and the compactive effort sufficient to achieve a density on the order of 95 to 100 percent of standard Proctor.

Significant data has been developed in the last decade to define more narrowly the anticipated range of strength reduction as a function of geosynthetic type and average grain size of backfill used (D_{50}).[1] Since the source of backfill within the reinforced zone is often unknown during design, the choice must usually be made on the basis of the specification characteristics (maximum size and average size) of the backfill permitted by the specifications.

Geosynthetics can be conveniently grouped for the determination of installation damage as either (1) uniaxial high density polyethylene (HDPE) geogrids, (2) biaxial polypropylene (PP) geogrids, (3) PVC coated high tenacity polyester (PET) geogrids, (4) Acrylic coated high tenacity polyester (PET) geogrids, (5) woven PET and polypropylene (PP) geotextiles, (6) non woven PET and PP geotextiles, and (7) woven PP slit film geotextiles. Geotextiles, if used for reinforcement applications, must meet the minimum requirements for Class 1 AASHTO M-288-96 and have a minimum weight of 270 g/m^2.

The larger reduction factor in each range in Table 1 is consistent with the lowest weight of geotextile available (150-200 g/m^2) or the lowest strength geogrid available within each group. It appears that only slit film geotextiles may have reduction factors for construction damage equal to or greater than the AASHTO default value of 3.0 if constructed with angular gravel fills. Further, considerable savings can be affected for load carrying applications by specifying fill with a maximum size of 20 mm and geotextiles with a mass greater than 270 g/m^2.

Based on the available data to date, we believe the following range of reduction factors can be used to supercede the guidance contained in the 1992 AASHTO Commentaries:

Table 1. Installation Damage Reduction Factors

Reduction Factor, RF_{ID}			
No.	Geosynthetic	Type 1 Backfill Max. Size 102 mm D_{50} about 30 mm	Type 2 Backfill Max. Size 20 mm D_{50} about 0.7 mm
1	HDPE uniaxial geogrid	1.20 - 1.45	1.10 - 1.20
2	PP biaxial geogrid	1.20 - 1.45	1.10 - 1.20
3	PVC coated PET geogrid	1.30 - 1.85	1.10 - 1.30
4	Acrylic coated PET geogrid	1.30 - 2.05	1.20 - 1.40
5	Woven geotextiles (PP & PET)	1.40 - 2.20	1.10 - 1.40
6	Non woven geotextiles (PP & PET)	1.40 - 2.50	1.10 - 1.40
7	Slit film woven PP geotextiles	1.60 - 3.00	1.10 - 2.00

Aging Reduction Factor, RF_D

Aging strength losses in any polymeric material are a result of molecular chain scission which reduces the molecular weight of the product. This mechanism is initiated in polyolefin product (PP and HDPE) by oxidation (exposure to oxygen), heat, exposure to Ultra Violet (UV) radiation and accelerated by the presence of high levels of soluble transition metals in the soil. In polyester (PET) products it is initiated by a process of hydrolysis or surface erosion in any aqueous environment and is accelerated by temperature, the reduction factors are temperature dependent. For in-ground use, the design temperature is taken as 20°C, although it has been measured as high as 35°C within 2 meters of the surface in tropical climates.

A. Polyester geosynthetics

To mitigate strength loss from contact or immersion in aqueous solutions, PET products can be manufactured with a high number molecular weight, M_n, and a low Carboxyl End Group (CEG) number; and should not be used in highly acidic or alkaline regimes. Note that PET tends to absorb and retain water from its environment, and will tend to remain saturated even above the water table from infiltrating surface runoff in moderate rainfall locations.

Typically, high tenacity fibers used for woven geotextiles and coated geogrid products are produced with a $M_n >$ than 25,000 and a CEG < 30 qualifying them as more resistant to hydrolysis. By contrast, low tenacity non woven geotextiles are currently produced with a M_n < than 20,000 and a CEG > 45 making them less resistant to hydrolysis. Testing protocols to measure strength loss due to hydrolysis and surface erosion in various aqueous solutions have been developed under an FHWA research program on Geosynthetic Durability.[2]

Based on the developed data, PET geosynthetics are recommended for use in environments characterized by $3 < pH < 9$, only. The following reduction factors for PET aging (RF_D) are presently indicated for a 100 year design life in the absence of product specific testing.

Table 2. Aging Reduction Factors, PET

No.	Geosynthetic	Reduction Factor, RF_{ID}	
		$5 < pH < 8$	$3 < pH < 5$ $8 < pH < 9$
1	Geotextiles $M_n < 20,000$, $40 < CEG < 50$	1.6	2.0
2	Coated geogrids $M_n > 25,000$, $CEG < 30$	1.15	1.3

* Use of materials outside the indicated pH or molecular property range requires specific product testing.

B. Polyolefin geosynthetics

To mitigate thermal and oxidative degradative processes, polyolefin products are stabilized by the addition of antioxidants for both processing stability and long term functional stability. These antioxidant packages are proprietary to each manufacturer and their type, quantity and effectiveness varies. Without residual antioxidant protection (after processing), PP's are vulnerable to oxidation and significant strength loss within a projected 75 to 100 year design life at 20°C. Current data suggests that unstabilized PP has a half life of less than 50 years.

Therefore the anticipated functional life of a PP geosynthetic is, to a great extent, a function of the type and remaining antioxidant level, and the rate of subsequent antioxidant consumption.

Antioxidant consumption is related to the oxygen content in the ground, which is typically less than atmospheric.

HDPE is less vulnerable to the same process, and substantial in-ground history has been developed by the pipe industry which has documented successful in-ground use for over 50 years using similar polymer formulation and antioxidants. Testing protocols to measure the time over which antioxidants are effective have been developed under an FHWA research program on Geosynthetic Durability for certain polyolefin products.[(2)]

At present, heat aging protocols for PP products, at full or reduced atmospheric oxygen, with subsequent numerical analysis are available for PP products which exhibit no initial cracks or crazes in their as manufactured state, typically monofilaments.[(3)] For PP products with initial crazes or cracks, typically tape products, or HDPE, heat aging testing protocols may change the nature of the product and therefore may lead to erroneous results. Alternate testing protocols using oxygen pressure as a time accelerator are under study and development.[(2)]

Since each product has a unique and proprietary blend of antioxidants, product specific testing is required to determine the effective life span of protection at the in-ground oxygen content. Limited data suggests that certain antioxidants are effective for 100 years in maintaining strength for in-ground use, where a reduced oxygen regime is likely.

A rough measure of antioxidant effectiveness for PP products formulated without significant carbon black is resistance to UV degradation measured in accordance with ASTM D 4355. A retained strength of 90% at 500 hours or more generally indicates an effective antioxidant blend and potentially a reduction factor as low as 1.1 at 20°C and 100 years. For HDPE geogrids presently available (Tensar UX series), current research data indicates a Reduction Factor of 1.1 for use at 20°C and 100 years.

Conclusion:
Design service life prediction for geosynthetics is no longer an unknown quantity requiring ultra-conservative safety factors or default values in building codes. Results of scientific testing are now available that can be used to estimate long term durability values for geosynthetic materials. When first confronted with a number of new geotechnical materials, AASHTO and FHWA agreed on a very prudent or conservative set of numbers to minimize risk while we learned more about the performance of these geotechnical materials. We have now studied and tested these materials and are now able to recommend the new reduction factors listed in Tables 1 and 2. These factors result in significant cost savings over the previous empirically developed factors.

Recommendation:
Design engineers should no longer be reluctant to specify geosynthetic materials in reinforcement applications due to the lack of poor durability performance information. Recent laboratory test results and field experience have shown that these materials are durable and information is readily available to predict design service life. The values listed in this Technical Note should be used in place of current AASHTO default values.

References:
1. V. Elias, Corrosion/Degradation of Soil Reinforcements for Mechanically Stabilized Earth Walls and Reinforced Soil Slopes, October 1996 Draft, FHWA SA-96-072.
2. Durability of Geosynthetics for Highway Applications, Interim Draft Report, Tasks D & E, DTFH 61-91-R-00054.
3. A. Salman, V. Elias, et al., Durability of Geosynthetics Based on Accelerated Laboratory Testing, Geosynthetics '97, San Diego, CA.

For further information, please contact a member of the Geotechnology Outreach Team: Richard Cheney and/or Jerry DiMaggio, HNG-31; Al DiMillio and/or Carl Ealy, Mike Adams, HNR-10.

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Appendix G
RECENT ROADWAY RESEARCH

The results of large-scale laboratory experiments on geosynthetic reinforced flexible pavements have been reported by Barksdale et al. (1989) and Chan et al. (1989). Test sections consisted of thin (25 to 38 mm) asphaltic concrete surfaces on 150 or 200 mm-thick aggregate bases over soft silty clay subgrades (CBR about 2.5). The sections were subjected to moving wheel loads. It was found that for pavements designed to carry more than about 200,000 equivalent 80-kN single-axle loads (SAL), the effect of geosynthetic reinforcement was relatively small because it did not significantly influence overall pavement stiffness. It was also found that prerutting of especially high-quality aggregate base densified the aggregate and led to greater rut resistance than either prestressing the geosynthetic reinforcement or using of stiff, nonprestressed reinforcement.

An open-mesh-type geogrid was found to have the same reinforcing capability as a woven geotextile, even though the geotextile had a 5% secant modulus 2.5 times greater than the geogrid. Minimum recommended geosynthetic modulus for aggregate base reinforcement is 260 kN/m for grids and 700 kN/m for woven geotextiles (Barksdale et al., 1989).

It was found that light- to moderate-strength pavement sections (AASHTO structural numbers, SN $\leq$ 2.5 to 3) placed on weak subgrades (CBR $\leq$ 3; $M_R < 24$ MN/m^2) are most likely to benefit from geosynthetic reinforcement. For these conditions, and with the geosynthetic moduli mentioned above, reductions in base course thicknesses of 10 to 20% (corresponding to 25 to 50 mm) are possible. For weaker subgrades, improvement is likely to be even greater, especially with thinner pavements and low-quality aggregates.

The quality of the base course aggregate was also found to influence performance of the pavement. Rutting of low-quality aggregate base can be reduced with geosynthetic reinforcement. In this case, the reinforcement should be placed within the base to minimize rutting, particularly on stronger subgrades. On weaker subgrades, the reinforcement should be placed at or near the bottom of the base.

Even if geosynthetic reinforcement is used in the roadway, Barksdale et al. (1989) correctly emphasizes that good construction practice and quality control procedures are still required. The subgrade should be properly prepared and proofrolled and the base course aggregate should be high quality with minimum fines content and compaction to 100% Modified Proctor.

A full-scale test of geogrid reinforced pavements on poor (CBR ~ 4) to fair (CBR ~ 6) clay subgrades was reported by Miuara, et al. (1990). They found that test sections with the reinforcement placed at the bottom of the base out-performed sections with geogrids placed near the middle of the aggregate base. The stronger grid was better than the weaker grid. Although the unreinforced control section experienced greater ruts and settlement, it had less surface cracking than the reinforced sections.

Finally, an important study on geogrid-reinforced flexible pavements was carried out at the USAE Waterways Experiment Station (Webster, 1993). Large-scale trafficking tests were performed on reinforced and unreinforced sections with different aggregate base thicknesses on compacted silty clay subgrades of CBR = 3 and 8. For such subgrades, no separation effect was expected nor was any observed. The primary reinforcement mechanisms were found to be:

1. geogrid interlock with the aggregate base,
2. subgrade confinement, which tended to limit the amount of upheaval of the subgrade into the base course, and
3. the tensioned membrane effects.

This latter effect is somewhat speculative, since measurements of the strains in the geogrids were not made. Some types of geogrids performed better than others. Relatively stiff, sheet-like grids were found to be the best, while only marginal improvements in performance were observed with a coated multifilament polyester grid. As with previous research, the best location for the reinforcement was found to be at the interface between the base course and the subgrade. A design procedure for flexible pavements for light aircraft was presented by Webster (1993) and is based on the results of the full-scale tests. Using rather stiff geogrids, thickness reductions of between 5 and 40% in total pavement section (AC + granular base) are possible, with the larger values applicable to thinner sections.

Finally, Koerner and Koerner (1994) make a distinction between the functions of separation, stabilization, and reinforcement, based on subgrade strength. They consider subgrade strengths of CBR $\leq$ 3 as *very poor* and good candidates for geosynthetic reinforcement.

The function of separation is secondary and often not considered for temporary roads, while filtration is probably important. Subgrades with 3 < CBR < 7 are considered *intermediate*, and the geotextile functions of separation and filtration become important as the need for reinforcement decreases. Subgrades with CBR > 7 are termed *fair*, and separation is the primary geotextile function. For reasons discussed in Section 5.1-2, we are skeptical of the need for geosynthetics in subgrades with strengths much greater than CBR of about 3 (undrained shear strength ≈ 90 kPa). Unless the aggregate base is exceptionally thin (< 150 mm) and the traffic extremely heavy and long-lived, in our opinion, it is unlikely that a geosynthetic will be very useful in such soils.

Koerner, R.M. and Koerner, G.R., *Separation: Perhaps the Most Under-Estimated Geotextile Function*, Geotechnical Fabrics Report, Vol. 12, No. 1, 1994, pp. 4-10.

Appendix H
REPRESENTATIVE[1] LIST OF GEOSYNTHETIC MANUFACTURERS AND SUPPLIERS

Akzo Nobel Industrial Systems
Ridgefield Business Center
Suite 318, Ridgefield Court
Asheville, NC 28802
(704) 665 - 5050

Amoco Fabrics and Fibers Co.
900 Circle 75 Parkway, Suite 300
Atlanta, GA 30339
(404) 984 - 4444

Bayex Inc.
14770 East Ave.
P.O. Box 390
Albion, NY 14411-0390
(800) 263 - 5715

Claymax Corporation
P.O. Box 88
Fairmount, GA 30139
(706) 337 - 5316

Environmental Protection, Inc.
P.O. Box 333
Mancelona, MI 49659-0333
(616) 587 - 9108

GSE Lining Systems
19103 Gundle Road
Houston, TX 77073
(800) 435 - 2008

National Seal Company
1245 Corporate Blvd
Aurora, IL 60504
(708) 898 - 1161

JPS Elasomerics Corporation
395 Pleasant St.
Northhampton, MA 01060
(413) 586 - 8750

LINQ Industrial Fabrics Inc.
2550 West 5th North Street
Summerville, SC 29483
(800) 543 - 9966

Seaman Corporation
1000 Venture Blvd
Wooster, OH 44691
(615) 691 - 9476

Nicolon Corporation
3500 Parkway Lane, Suite 500
Norcross, GA 30092
(800) 234 - 0484

Palco Lining Inc.
2624 Hamilton Blvd
S. Plainfield, NJ 07080
(908) 898 - 6262

Spartan Technologies
P.O. Box 1658
Spartanburg, SC 29304
(800) 638 - 1843

Poly-Flex, Inc.
2000 West Marshall Drive
Grand Prairie, TX 75051
(214) 647 - 4374

Reemay Inc.
70 Old Hickory Blvd.
Old Hickory, TN 37138
(800) 321 - 6271

Tenax Corporation
4800 East Monument Street
Baltimore, MD 21205
(410) 522 - 7000

Serrot Corporation
8383 Cassia Way
Henderson, NV 89014
(702) 566 - 4739

Synthetic Industries
Construction Products Division
4019 Industry Drive
Chattanooga, TN 37416
(800) 621 - 0444

Tensar Earth Technologies
5775-B Glenridge Drive
Suite 450, Lakeside Center
Atlanta, GA 30328
(800) 836 - 7271

Staff Industries, Inc.
240 Chene Street
Detroit, MI 48207
(800) 526 - 1368

Watersaver Company, Inc.
P.O. Box 16465
Denver, CO 80216-0465
(303) 289 - 1818

1. List is from the Industrial Fabrics Association International, Geotextile and Geomembrane Divisions membership lists.

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Appendix I
GENERAL PROPERTIES AND COSTS OF GEOTEXTILES AND GEOGRIDS

TABLE I1 - GENERAL RANGE OF STRENGTH AND PERMEABILITY PROPERTIES[1, 2] FOR REPRESENTATIVE TYPES OF GEOTEXTILES AND GEOGRIDS

Geotextile Type	Weight [3] (g/m^2)	Ultimate Tensile[4] Strength (kN/m)	Strain at Ultimate[4] Tensile Strength (%)	Secant[4] Modulus at 10% Strain (kN/m)	Grab[5] Strength (N)	Puncture[6] Strength (N)	Burst[7] Strength (kPa)	Tear[8] Strength (N)	Equivalent[9] Darcy Permeability (m/sec)
Woven									
Monofilament-Polypropylene	120 - 240	16 - 70	20 - 40	70 - 260	700 - 2300	320 - 700	2700 - 4800	200 - 440	10^{-4} - 10^{-2}
Silt-Film	50 - 170	12 - 45	20 - 40	50 - 260	320 - 1600	80 - 600	1400 - 4800	200 - 1600	10^{-4} - 10^{-3}
Fibrillated Tape and Multifilament Polypropylene	240 - 760	35 - 210	15 - 40	175 - 700	700 - 6200	700 - 1100	4100 - 10400*	440 - 1800	10^{-4} - 10^{-3}
Multifilament-Polyester	140 - 710	25 - 350*	10 - 30	175 - 1050*	700 - 9000*	200 - 1400	3400 - 10400*	360 - 2300	10^{-4} - 10^{-3}
Nonwoven									
Continuous Filament-Melt Bonded	50 - 240	4 - 35	30 - 100	18 - 90	180 - 1800	80 - 440	550 - 3500	120 - 900	10^{-4} - 10^{-2}
Needlepunched (lightweight)	70 - 240	4 - 18	40 - 150	2 - 25	180 - 1100	200 - 550	1000 - 2700	120 - 700	10^{-3} - 10^{-2}
Needlepunched (heavyweight)	240 - 850	8 - 35	40 - 150	9 - 55	700 - 2300	440 - 1100	2000 - 6900	320 - 900	10^{-4} - 10^{-2}
Geogrid									
Polypropylene	140 - 240	8 - 35	10 - 20	90 - 230	n/a	n/a	n/a	n/a	> 10
High-Density Polyethylene	240 - 710	8 - 90	10 - 20	55 - 70	n/a	n/a	n/a	n/a	> 10
Polyester	240 - 710	35 - 140	5 - 15	350 - 2600	n/a	n/a	n/a	n/a	> 10

1. Data was obtained from numerous sources, in some cases estimated, and represents an average range. There may be products outside this range. No relation should be inferred between maximum and minimum limits for different tests.
2. Both directions
3. Method 1.1.84, Appendix B, FHWA Geotextile Engineering Manual
4. Wide Width Method, ASTM D-4595
5. ASTM D-4632
6. ASTM D-4833
7. ASTM D-3786
8. ASTM D-4533
9. ASTM D-4491

* Limited by test machine

TABLE I2
GENERAL DESCRIPTION OF GEOTEXTILES

Property	Description		
	LOW	MODERATE	HIGH
Tensile Strength (kN/m)	< 15	15 - 50	> 50
ϵ @ ult.	< 20%	20% - 50%	> 50%
Burst (kPa)	< 1400	1400 - 3400	> 3400
Permeability, k (m/s)	< 0.0001	0.0001 - 0.001	> 0.001
Cost ($/m^2)	< $1	$1 - $2	> $2

TABLE I3
APPROXIMATE COST[1,2] RANGE OF GEOTEXTILES AND GEOGRIDS

Geosynthetic	Material Cost[1,2] ($/m^2)
Filtration Geotextiles - high survivability	1.25 - 1.75
Erosion Control Mats	3.50 - 6.00
Temporary Erosion Control Blankets	1.25 - 2.50
Roadway Geotextile Separators - high survivability	1.25 - 1.75
Asphalt Overlay Geotextiles	0.60 - 1.25
Geotextile Embankment Reinforcement[3]	2.50 - 12.00
Geogrid/Geotextile Wall and Slope Reinforcement[4,5] - per 15 kN/m long-term allowable strength	1.50 - 3.50

NOTES:
1. **Typical** costs for materials delivered on-site, for use in Engineer's estimate. Costs are exclusive of installation and Contractor's markup.
2. Installation cost of geosynthetics **typically** cost $0.30 to $0.90, except for very soft ground and underwater placement.
3. Assumes design strength is based upon a 5% to 10% strain criteria with an ASTM D 4595 test.
4. Assumes allowable design strength is based upon a complete evaluation of partial safety factors, as detailed in Appendix J.
5. Material costs of $14.00 to $20.00 should be anticipated if the alternative procedure for determination of long-term design strength (Appendix J, section J.5) is used.

Appendix J

GRI SURVEY OF STATES

(as of December 31, 1992)

Application	Assessment Category	A[1]	B[1]	C[1]	D[1]	E[1]	F[1]
Separation	States	1	4	19	23	1	4
	Percent	2	8	36	44	2	8
Separation/Stabilization[2]	States	-	-	25	21	2	4
	Percent	-	-	48	40	4	8
Filtration	States	4	18	15	10	1	4
	Percent	8	34	29	19	2	8
Erosion Control	States	5	17	19	5	2	4
	Percent	10	33	36	9	4	8
Sediment Control	States	5	3	15	24	1	4
	Percent	10	6	28	46	2	8
Reflective Cracking	States	5	10	0	33	0	4
	Percent	10	19	0	63	0	8

1. A = follows AASHTO/Task Force 25 exactly
   B = follows AASHTO/Task Force 25 closely
   C = individual State table uses other values
   D = topic not addressed in specifications
   E = State prequalifies geotextiles
   F = not available for survey
2. This category is not specifically addressed in AASHTO/Task Force 25 recommendations; however, it is present in many State specifications and is included for that reason.

Geosynthetic Research Institute Report #7, Drexel University, Philadelphia, PA, December 1992.

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Appendix K
GEOSYNTHETIC REINFORCEMENT STRUCTURAL DESIGN PROPERTIES

Table of Contents

K.1 BACKGROUND

The tensile properties of geosynthetics are affected by environmental factors such as creep, installation damage, aging, temperature, and confining stress. Therefore, the long-term tensile strength, T_{al}, should be determined by thorough consideration of allowable elongation, creep potential and all possible strength degradation mechanisms.

An inherent advantage of geosynthetics is their longevity in fairly aggressive soil conditions. The anticipated half-life of some geosynthetics in normal soil environments is in excess of 1000 years. However, as with steel reinforcements, strength characteristics must be adjusted to account for potential degradation in the specific environmental conditions, even in relatively neutral soils.

Polymeric reinforcement, although not susceptible to corrosion, may degrade due to physicochemical activity in the soil such as hydrolysis, oxidation, and environmental stress cracking depending on polymer type. In addition, these materials are susceptible to installation damage and the effects of (potential) high temperatures at the facing and connection of retaining wall structures. Temperatures can be as high as 50°C compared with the normal range of in-ground temperature of 12°C in cold and temperate climates to 30°C in arid desert climates.

Degradation most commonly occurs from mechanical damage, long-term time dependent degradation caused by stress (creep), deterioration from exposure to ultraviolet light, and chemical or biological interaction with the surrounding environment. Because of varying polymer types, quality, additives and product geometry, each geosynthetic is different in its resistance to aging and attack by different chemical and biological agents. Therefore, each product must be investigated individually.

This appendix is broken into several categories, to aid the user. First, the partial reduction factor methodology for determining long-term strength is presented. Next, application of long-term strength to allowable (*i.e.*, design) strength is reviewed for wall and slope applications. Then, a discussion on use of the partial reduction factor method and evaluation of supplier submittals by agencies is presented. Implementation obstacles are noted within this discussion. An alternative procedure, less cumbersome to implement, for determination of a long-term strength is then presented. This simplified procedure is for complementary use with the more-detailed methodology, and is presented as a means to encourage use of geosynthetics in reinforced soil structures.

K.2 TENSILE STRENGTHS

K.2-1 Long-Term Tensile Strength

Long-term tensile strength (T_{al}) of the geosynthetic shall be determined using a partial factor approach (Bonaparte and Berg, 1987). Reduction factors are used to account for installation damage, chemical and biological conditions and to control potential creep deformation of the polymer. The total reduction factor is based upon the mathematical product of these factors. The long-term tensile strength, T_{al}, thus can be obtained from:

$$T_{al} = \frac{T_{ult}}{RF} \qquad \text{[K-1]}$$

with RF equal to the product of all applicable reduction factors:

$$RF = RF_{CR} \times RF_{ID} \times RF_{D} \qquad \text{[K-2]}$$

where:

T_{al} = long-term geosynthetic tensile strength, (kN/m);
T_{ult} = ultimate geosynthetic tensile strength, based upon MARV, (kN/m);
RF_{CR} = creep reduction factor, ratio of T_{ult} to creep-limiting strength, (dimensionless);
RF_{ID} = installation damage reduction factor, (dimensionless); and
RF_{D} = durability reduction factor for chemical and biological degradation, (dimensionless).

K.2-2 Allowable Strength

Additionally, the following factors should be considered. The *long-term* strength determined by dividing the ultimate strength by RF does not include an overall factor of safety to account for variation from design assumptions (*e.g.*, heavier loads than assumed, construction placement, fill consistency, etc.). Therefore, an *allowable* strength needs to be defined and quantified. If seams or joints in the reinforcement are allowed, they likely will reduce the strength of the material, and an applicable reduction factor should be included in computation of an allowable strength. Thus the allowable strength of a geosynthetic for RSS applications can be defined as:

$$T_{a} = \frac{T_{al}}{FS \times RF_{JNT}} \qquad \text{[K-3]}$$

where:

- T_a = allowable geosynthetic tensile strength,(kN/m);
- FS = overall factor of safety to account for uncertainties in the geometry of the structure, fill properties, reinforcement properties, and externally applied loads; and
- RF_{JNT} = partial factor of safety for joints (seams and connections), (dimensionless).

For MSE wall structures, a minimum factor of safety, FS, of 1.5 is recommended. Of course, the FS value should be dependent upon the specifics of each project.

For RSS structures, the FS value will be dependent upon the analysis tools utilized by the designer. For most design charts, a FS is incorporated into the soil shear strength and a FS = 1 should be used in Eq. L-3 to determine the allowable geosynthetic strength. With computerized analyses, the FS value is dependent upon how the specific program accounts for the reinforcement tension is computing a stability factor of safety.

The FHWA's *RSS* program, and many others, assume the reinforcement force as contributing to the resisting moment in the computation of a stability factor of safety, *i.e.*:

$$FS_{STABILITY} = \frac{M_R + T_S R}{M_D} \qquad \text{[K-4]}$$

With this assumption, the allowable strength is equal to the long-term strength (if no reduction is required for seams or joints). The factor of safety on the reinforcement is equal to the stability factor of safety. Therefore, a value of FS = 1 is used in Eq. K-3.

Some computer programs use an assumption that the reinforcement force is a negative driving component, thus the stability FS is computed as:

$$FS_{STABILITY} = \frac{M_R}{M_D - T_S R} \qquad \text{[K-5]}$$

With this assumption, the stability factor of safety is not applied to T_S. Therefore, the allowable strength should be computed as the long-term strength divided by the safety factor (*e.g.*, target stability factor of safety). A FS value of 1.3 to 1.5 is typically used in Eq. K-3, in this case.

K.3 REDUCTION FACTORS

K.3-1 Ultimate Strength

Ultimate strength values shall be based upon minimum average roll values (MARV), as defined in ASTM D 4759. Ultimate strength for agency quality assurance (QA) purposes may be determined according to ASTM D 4595 Test Method for Tensile Properties of Geotextiles by the Wide-Width Strip Method or GRI:GG1 Geogrid Single Rib Tensile Strength. The test procedure used to determine ultimate strength, however, must be the same as that used to define RF_{CR}.

K.3-2 Creep

The creep reduction factor, RF_{CR}, is the ratio of the ultimate strength, T_{ULT}, to the creep limited strength obtained from laboratory creep tests for each product. Typical ranges of reduction factors as a function of polymer type, are summarized in Table K-1.

TABLE K-1
TYPICAL RANGES OF CREEP REDUCTION FACTORS

Polymer Type	Creep Reduction Factor
Polyester	2.5 to 2.0
Polypropylene	5 to 4.0
Polyethylene	5 to 2.5

The long-term tension-strain-time polymeric reinforcement behavior shall be determined from results of controlled laboratory creep tests conducted for a minimum duration of 10,000 hours for a range of load levels on samples of the finished product. At present, creep tests are conducted in-isolation rather than confined in soil, even though in-isolation creep tests tend to over predict creep strains and under predict the true creep strength when used in a structure. Creep testing should be conducted in accordance with ASTM D 5262, Test Method for Evaluating the Unconfined Tension Creep Behavior of Geosynthetics. For confined creep testing (NOTE: a standardized test procedure is not available), project-specific or representative backfill material and typical placement and confinement conditions should be in the testing.

The creep reduction factor is determined by comparing the long-term creep strength, T_l, to the average ultimate tensile strength of the samples tested for creep. The samples tested for ultimate strength should be taken from the same lot, and preferably the same roll, of material which is used for the creep testing. For ultimate limit state design, the strength reduction factor to prevent long-term creep rupture is determined as follows:

$$RF_{CR} = \frac{T_{ultlot}}{T_l} \qquad [K\text{-}6]$$

where,

T_{ultlot} = the average lot specific ultimate tensile strength for the lot of material used for creep testing; and

T_l = the long-term creep rupture strength at the design life and design temperature.

Creep test data at a given temperature may be directly extrapolated over time up to one order of magnitude, in accordance with standard polymeric practices. Considering that typical design lives for permanent MSE wall and RSS structures are 75 years or more, extrapolation of creep data is required. Therefore, accelerated testing (*e.g.*, testing under elevated temperatures or possibly other corroborating evidence) is required to extrapolate 10,000-hour creep test data to a minimum 75-year design life. Accelerated testing is used to extrapolate to a 75-year design life and to ensure that the rupture failure mechanism, does not change (*e.g.*, transition from ductile to brittle rupture). A step-by-step procedure for extrapolating and interpolating stress rupture data in presented in Appendix B of Elias and Christopher (1997).

Additional judgement should be exercised when selecting an creep limited value for geosynthetics whose creep response is established with confined (in-soil) creep testing. Field performance of structures designed on the basis of confined creep test data is limited. However, it is well-established and generally recognized that unconfined creep test results are consistently conservative.

The requirement for a 10,000-hour minimum creep test period for geogrids and geotextiles may be waived for a new product if it can be demonstrated that it is sufficiently similar to a proven 10,000 hour creep tested product of a similar nature. Product similarity must consider base resin, resin additives, product manufacturing process, product geometry, and creep response. Creep testing of all products, regardless of similarity, shall be conducted for a least 1,000 to 2,000 hours. See Elias and Christopher (1997) for details on use of creep data from similar products.

K.3-3 Installation Damage

The installation damage factor reduces the long-term strength to account for the effect of installation damage on geosynthetic reinforcement. RF_{ID} can range from 1.05 to 3.0, and is a function of the backfill gradation, compaction techniques, product structure, and product mass per unit area. Installation damage factors shall be determined from the results of full-scale construction damage tests. Damaged samples should be obtained following ASTM D 5818, Practice for Obtaining Samples of Geosynthetics from a Test Section for Assessment of Installation Damage. This practice requires that the geosynthetic material is subjected to a backfilling and compaction cycle, consistent with field practice. The ratio of the initial reinforcement strength to the strength of the retrieved samples defines the RF_{ID}.

For reinforcement applications a minimum weight of 270 g/m^2 for geotextiles is recommended to minimize installation damage. This roughly corresponds to a Class 1 geotextile as specified in AASHTO M288.

See Table F-1 for typical ranges of installation damage factors, by geosynthetic type.

K.3-4 Durability

The durability reduction factor, RF_D, is dependent on the susceptibility of the geosynthetic to attack by microorganisms, chemicals, thermal oxidation, hydrolysis and stress cracking, and can vary typically from 1.1 to 2.0. Because of varying polymer types, quality, additives and product geometry, each geosynthetic is different in its resistance to aging and attack by different chemical and biological agents. Therefore, each product must be investigated individually.

Typically, polyester products (PET) are susceptible to aging strength reductions due to hydrolysis (water availability) and high temperatures. Hydrolysis and fiber dissolution are accelerated in alkaline regimes, below or near piezometric water levels or in areas of substantial rainfall where surface water percolation or capillary action ensures water availability over most of the year.

Polyolefin products (PP and HDPE) are susceptible to aging strength losses due to oxidation (contact with oxygen) and or high temperatures. The level of oxygen in reinforced fills is a function of soil porosity, ground water location and other factors not yet fully understood. However, it is considerably less than oxygen levels in the atmosphere (21 percent). Therefore, oxidation of geosynthetics in the ground should proceed at a slower rate than those used above ground. Oxidation is accelerated by the presence of transition metals (Fe, Cu, Mn, Co, Cr) in the backfill as found in acid sulphate soils, slag fills, other industrial wastes or mine tailings containing transition metals. It should be noted that the resistance of polyolefin geosynthetics to oxidation is primarily a function of the proprietary antioxidant package added to the base resin, which differs for each product brand, even when formulated with the same base resin.

The artificial relative resistance of polymers to these identified regimes is shown in Table K-2. Polymeric reinforcements may, therefore, be chosen consistent with the preliminary data shown in Table K-2. Limits, based upon current research, on the pH limits of the reinforced fill, by polymer type, are listed in Table K-3. Specific aging reduction factor values are discussed in Appendix F.

Most geosynthetic reinforcement is buried, and therefore ultraviolet (UV) stability is only of concern during construction and when the geosynthetic is used to wrap the wall or slope face. If used in exposed locations, the geosynthetic should be protected with coatings or facing units to prevent deterioration. Vegetative covers can also be considered in the case of open weave geotextiles or geogrids. Thick geosynthetics with ultraviolet stabilizers can be left exposed for several years or more without protection; however, long-term maintenance should be anticipated because of both UV deterioration and possible vandalism.

TABLE K-2
ANTICIPATED RESISTANCE OF POLYMERS TO SPECIFIC ENVIRONMENTS

Soil Environment	Polymer		
	PET	PE	PP
Acid Sulphate Soils	NE	?	?
Organic Soils	NE	NE	NE
Saline Soils pH < 9	NE	NE	NE
Calcareous Soils	?	NE	NE
Modified Soils/Lime, Cement	?	NE	NE
Sodic Soils, pH > 9	?	NE	NE
Soils with Transition Metals	NE	?	?

NE = No Effect
? = Questionable Use, Exposure Tests Required

TABLE K-3
RECOMMENDED PH LIMITS FOR REINFORCED FILL SOILS

Base Polymer	Test Method	Limits
Polyester (PET)	AASHTO T289	>3 and < 9
Polyolefin (PP & HDPE)	AASHTO T289	>3

The protocols for testing to obtain this reduction factor are under development. In general, it consists for oven aging polyolefins (PP and HDPE) samples to accelerate oxidation and measure their strength reduction, as a function of time, temperature and oxygen concentration. This high temperature data must then be extrapolated to a temperature consistent with field conditions. For polyesters (PET) the aging is conducted in an aqueous media at varying pH's and relatively high temperature to accelerate hydrolysis, with data extrapolated to a temperature consistent with field conditions.

K.3-5 Joints, Seams, and Connections

The effect of the joint strength must be factored into design strength when separate lengths of geosynthetics are connected together or overlapped in the direction of primary force development. The value of FS_{JNT} should be taken as the ratio of the unjointed specimen strength to the joined specimen strength. Testing should be conducted in accordance with ASTM D 4595 for mechanically connected joints and GRI:GG5 or GRI:GT6 for overlap joints. Sustained tension tests of 1,000-hour minimum duration should also be conducted on mechanically connected joints, according to GRI:GG4 and GRI:GT7. A load level equal to the allowable strength, T_a, is suggested for long-term testing. Limits on number and location of joints and seams in a slope or wall structure should be addressed in the project specifications.

As discussed in Chapter 9, MSE Retaining Walls and Abutments, the connection geosynthetic strength to the wall facing element may limit strength and, therefore, control the allowable design tensile strength. Connection strength must be addressed in wall designs. Connection strength requirements, testing, and test data interpretation are addressed in Elias and Christopher (1997).

K.4 IMPLEMENTATION

The determination of the reduction factors for creep (including extrapolation) and chemical/biological durability require extensive testing and is product-specific. Testing standards for determination of these partial factors are not fully developed; thus, test procedures and/or result interpretation can vary. Therefore, evaluation of supplier submittals is not an easy process for many agencies.

Detailed lists of items to be supplied by potential geosynthetic reinforcement material suppliers and system suppliers, are presented in the *Guidelines for Design, Specification, & Contracting of Geosynthetic Mechanically Stabilized Earth Slopes on Firm Foundations*, U.S. Department of Transportation, Federal Highway Administration, FHWA-SA-93-025, (Berg, 1993).

Guidelines for assigning evaluation responsibilities to the various agency organizations are also provided in these guidelines. However, these guidelines are focused upon scenarios where agencies evaluate materials and use an approved products or vendor list system.

Implementation of geosynthetic RSS and MSE wall technologies has been significantly hampered by the review required to assess geosynthetic long-term allowable strength. When an approved products or vendor list is not used, many agencies find the procedure too laborious and time-consuming for post-bid evaluation of materials. Other agencies are not able to implement a complete review process, amongst their various organizational units, for a variety of reasons, or do not feel that they have the in-house expertise to complete an evaluation.

Impeded implementation of well-documented, cost-effective geosynthetic reinforced slope and wall technologies is costly. Funds saved by implementing these technologies can be well-spent on other transportation projects.

It is recognized that some agencies need an easier and/or quicker procedure for implementing long-term allowable strength quantification and geosynthetic product acceptance. Therefore, an alternative procedure for determining long-term strength is presented in Section K.5. This alternative procedure is to be used in conjunction with, and to compliment, the detailed procedure presented in Section K.2.

K.5 ALTERNATIVE LONG-TERM STRENGTH DETERMINATION

An alternative for computing a long-term allowable strength (Section K.2) is presented in this section. This alternative procedure is for complementary use with the more detailed procedure.

The goal of providing an alternative method is to foster widespread use of geosynthetic reinforced slopes and walls in transportation facilities. Specific objectives include:

- providing an easy-to-use method for determining design strengths;
- providing agencies with a method to generically specify geosynthetic reinforcement with a defined default allowable strength (through a defined default long-term strength and a design safety factor), that suppliers will be required to use unless a detailed evaluation of long-term strength on their specific products has been completed by the agency;
- providing long-term strengths that are conservative and economical;

- providing long-term strengths that are sufficiently punitive that thorough testing and evaluation of geosynthetic materials by manufacturers and suppliers is still promoted;
- providing an interim method until a sufficient number of materials and/or systems are approved by an agency and an approved list specifications is developed;
- providing a method for use with conservative soil environment parameters; and
- maintaining the current state-of-practice.

These objectives and goals can be achieved using a single default reduction factor to account for creep, installation damage, durability, and connections. A reasonable default reduction factor, RF, which meets the stated goal and objectives, is presented below.

For preliminary design of permanent structures and for applications defined by the user as not having severe consequences should poor performance or failure occur, the long-term tensile strength, T_{al}, may be evaluated without product specific data, as:

$$T_{al} = \frac{T_{ult}}{7} \qquad [K\text{-}7]$$

with the default RF equal to 7. This reduction factor RF=7, should be limited to projects where the project environment meets the following requirements:
- granular soils (sands, gravels) used in the reinforced fill;
- $4.5 \leq pH \leq 9$;
- site temperature < 30°C;
- biologically inactive environments; and
- maximum backfill particle size of 20 mm.

Other qualifiers on application of this default reduction factor is limiting use to projects where:
- maximum retaining walls height is 10 m;
- face element (for walls) shall be a nonaggressive environment for the geosynthetic;
- maximum reinforced slope height is 15 m;
- geotextile reinforcement meets AASHTO M 288 specification strength requirements for *High Survivability Level*; and
- the manufacturer certifies that the supplied geosynthetic is intended for and fit to use as long-term soil reinforcement.

Site temperature is defined as the temperature which is halfway between the average yearly air temperature and normal daily air temperature for the hottest month at the site.

The total reduction factor of 7 has been established by multiplying lower bound partial reduction factors obtained from currently available test data, for products which meet the minimum requirements in Table K-4. See Berg et al. (1998) for a discussion on the range of total reduction values used over the past twenty-five years.

It should be noted that the total Reduction Factor may be reduced significantly with appropriate test data. It is not uncommon for products with creep, installation damage and aging data, to develop total reduction factors in the range of 4 to 6.

Use of this alternative allowable strength procedure for structurally-faced, MSE retaining walls does not eliminate the requirement of connection strength testing. Testing shall be conducted to define the ultimate, short-term connection strengths, T_{ultc} (see Chapter 9).

TABLE K-4
MINIMUM REQUIREMENTS FOR USE OF DEFAULT REDUCTION FACTOR

Geosynthetic Type	Property	Test Method	Criteria to Allow Use of Default RF
Polypropylene	UV Oxidation Resistance	ASTM D 4355	Min. 70% strength retained after 500 hours in weatherometer.
Polyethylene	UV Oxidation Resistance	ASTM D 4355	Min. 70% strength retained after 500 hours in weatherometer.
Polyester	Hydrolysis Resistance	Intrinsic Viscosity Method (ASTM D 4603) with Correlation or Determine Directly Using Gel Permeation Chromatography	Min. Number (Mn) Molecular Weight of 25,000
Polyester	Hydrolysis Resistance	ASTM D 2455	Max. Carboxyl End Group Number of 30
All Polymers	Survivability (mass per unit area)	ASTM D 5261	Min. 270 g/m^2
All Polymers	% Post-consumer recycled material by weight	Certification of material used	Maximum 0%

For temporary applications not having severe consequences should poor performance or failure occur, a default value for RF of not less than 3 could be used. For more critical projects, a higher default reduction factor could be used.

Again, use of a default reduction factor, RF, is for complementary use with the more-detailed procedure. Blanket, long-term use of a default reduction factor will penalize many current suppliers and limit economic benefit of geosynthetic reinforced structures. Those manufacturers and suppliers that have or will conduct the extensive testing to document partial safety factors should be allowed to use those factors, after agency review and approval. Exclusive use of a blanket default value will also severely impede further evolution of this technology.

K.6 SOIL-REINFORCEMENT INTERACTION

Two types of soil-reinforcement interaction coefficients or interface shear strengths must be determined for design: pullout coefficient, and direct shear coefficient. Pullout coefficients are used in stability analyses to compute mobilized tensile force at the front and tail of each reinforcement layer in slopes and at the tail in walls. Direct shear coefficients are used to check factors of safety against outward sliding of the entire reinforced mass.

K.6-1 Pullout Resistance

Design of reinforced slopes requires evaluation of the long-term pullout performance with respect to three basic criteria:

i) pullout capacity; *i.e.*, the pullout resistance of each reinforcement should be adequate to resist the design working tensile force in the reinforcement with a specified factor of safety (FS_{PO}), where FS_{PO} is typically set at a minimum of 1.5; and

ii) allowable displacement; *i.e.*, the relative soil-to-reinforcement displacement required to mobilize the design tensile force should be smaller than the allowable displacement; and

iii) long-term displacement; *i.e.*, the pullout load should be smaller than the critical creep load.(Christopher et al., 1989)

The pullout resistance of the reinforcement is mobilized through one or a combination of two basic soil-reinforcement interaction mechanisms. The two mechanisms by which load may be transferred between soil and geosynthetic are: i) interface friction; and ii) passive soil resistance. Geotextile pullout resistance is developed with an interface friction mechanism. Geogrid pullout resistance may be developed by both interface friction and passive soil resistance against transverse elements. The load transfer mechanisms mobilized by a specific geogrid depends primarily upon its structural geometry (*i.e.*, composite reinforcement versus linear or planar elements, thickness of in-plane or out-of-plane transverse elements, and aperture dimension to grain-size ratio). The soil-to-reinforcement relative movement required to mobilize the design tensile force depends mainly upon the load transfer mechanism, the extensibility of the reinforcement material, and the soil type. (Christopher et al., 1989)

The long-term pullout performance (*i.e.*, displacement under constant design load) is predominantly controlled by the soil's creep characteristics and the reinforcement material. Soil reinforcement systems will generally not be used with cohesive soils susceptible to creep. Therefore, creep is primarily an issue of the reinforcement type. The basic aspects of pullout performance in terms of the major load transfer mechanism, relative soil-to-reinforcement displacement required to fully mobilize the pullout resistance, and creep potential of the reinforcement in granular (and low-cohesive) soils for generic extensible reinforcement types are presented in Table K-5. (Christopher et al., 1989)

Pullout resistance of geosynthetic reinforcement is defined by the lower value of:

i) the ultimate tensile load required to generate outward sliding of the reinforcement through the soil mass; or
ii) the tensile load which produces a 15 mm displacement as measured at the end of the embedded sample.

TABLE K-5
BASIC ASPECTS OF REINFORCEMENT PULLOUT PERFORMANCE IN GRANULAR AND LOW COHESIVE SOILS
(after Christopher et al., 1989)

Generic Reinforcement Type	Major Load Transfer Mechanism	Displacement to Pullout	Long-Term Performance
geogrids	frictional + passive H.D.	dependent on reinforcement extensibility (25 to 50 mm)	dependent on reinforcement structure and polymer creep
geotextiles	frictional (interlocking) L.D.	dependent on reinforcement extensibility (25 to 100 mm)	dependent on reinforcement structure and polymer creep
NOTE: L.D. - low-dilatency effect; H.D. - high-dilatency effect			

Several approaches and design equations have been developed and are currently being used to estimate pullout resistance by considering frictional resistance, passive resistance, or a combination of both. The design equations use different interaction parameters, therefore it is, difficult to compare the pullout performance of different reinforcements for a specific application (Christopher et al., 1989).

A normalized definition is recommended as presented in *FHWA Reinforced Soil Structures, Volume I - Design and Construction Guidelines* (Christopher et al., 1989). The ultimate pullout resistance, P_r, of the reinforcement per unit width of reinforcement is given by:

$$P_r = F^* \cdot \alpha \cdot \sigma'_v \cdot L_e \cdot C$$

where:

$L_e \cdot C$	=	the total surface area per unit width of the reinforcement in the resistance zone behind the failure surface, (m^2);
L_e	=	the embedment or adherence length in the resisting zone behind the failure surface, (m);
C	=	the reinforcement effective unit perimeter; *e.g.*, C=2 for geogrids and geotextiles, (dimensionless);
F^*	=	the pullout resistance (or friction-bearing-interaction) factor, (dimensionless);
α	=	a scale effect correction factor, (dimensionless); and
σ'_v	=	the effective vertical stress at the soil-reinforcement interfaces, (kN/m^2).

The pullout resistance factor, F*, can be most accurately obtained from pullout tests performed on the specific, or representative, backfill to be used on the project. Refer to GRI Test Method GG5 - Geogrid Pullout and GRI Test Method GT6 - Geotextile Pullout, as applicable, for pullout test procedures. Note that this test method produces pullout interaction coefficients that are classified as either short-term or long-term. **Design of MSE walls and slopes for permanent applications requires use of long-term interaction coefficients.**

For standard backfill materials, with the exception of uniform sands (i.e., coefficient of uniformity, $C_u < 4$), it is acceptable to use conservative default values for F* and α as shown in Table K-6.

TABLE K-6
DEFAULT VALUES FOR F* AND α PULLOUT FACTORS

Reinforcement Type	Default F*	Default α
Geogrid	0.8 tan ϕ	0.8
Geotextile	0.67 tan ϕ	0.6

Passive resistance is applicable only to geogrids, and not to geotextiles. The passive resistance portion of the above equation assumes that long-term passive resistance will occur across transverse geogrid ribs. This requires sufficient long-term junction strength between the transverse and longitudinal ribs to assure stress transfer. Long-term stress transfer is assured if the geogrid is creep-tested with the through-the-junction method per GRI:GG3a. Long-term pullout interaction coefficients should be quantified for **geogrids** with either:

i) quick, effective stress pullout tests and through-the-junction creep-testing of the geogrid per GRI:GG3a test method;
ii) quick, effective stress pullout tests of the geogrid with severed transverse ribs;
iii) quick, effective stress pullout tests of the entire geogrid structure if summation of shear strengths of the joints occurring in a 300 mm length of grid sample is equal to or greater than the ultimate strength of the grid element to which they are attached; or
iv) long-term effective stress pullout tests of the entire geogrid structure.

Long-term pullout interaction coefficients should be quantified for **geotextiles** with:

v) quick, effective stress pullout tests. (Berg, 1993)

Test Method a) *Controlled Strain Rate Method For Short-Term Testing* per GRI:GG5 and GRI:GT6 is recommended for testing under conditions i), ii), iii), and v) above. Test method d) Constant Stress (Creep) Method For Long-Term Testing per GRI:GG5 is recommended for condition iv) above. Joint shear strength shall be measured in accordance with GRI:GG2 and ultimate strength shall be measured with either GRI:GG1 or ASTM D 4595 for condition iii) above.

Long-term testing may also be required if cohesive soils are utilized, to define long-term effective stress (drained) pullout resistance. Procedures and results for long-term testing in cohesive soils have been presented by Christopher and Berg (1990). Their method is a combination of GRI:GG5 and GRI:GT6 methods c) Incremental Stress Method for Short-Term Testing, and d) Constant Stress (Creep) Method For Long-Term Testing.

K.6-2 Direct Shear Resistance

Soil-geosynthetic direct shear resistance should be determined in accordance with ASTM D 5321, Test Method for Determining the Coefficient of Soil and Geosynthetic or Geosynthetic and Geosynthetic Friction by the Direct Shear Method.

K.7 REFERENCES

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